Appendix E

Geotechnical Engineering Report

GEOTECHNICAL ENGINEERING REPORT

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GEOTECHNICAL ENGINEERING REPORT

High Bridge (SR-361.66) Replacement Project
Portageville, Wyoming and Livingston Counties, New York

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1.0 INTRODUCTION

Golder Associates Inc. (Golder) is pleased to submit this ***Geotechnical Engineering Report*** to Modjeski and Masters, Inc. (M&M) for the Norfolk Southern High Bridge (SR-361.66) Replacement Project, located about 0.75-miles northwest of the Village of Portageville in Wyoming and Livingston Counties, New York (see Figure 1).

Presented herein are Golder's findings, conclusions, and design recommendations with respect to the foundation and geotechnical engineering aspects of the subject project. In general, this report has been organized to provide the following information:

- A project description
- An overview of the regional and site geology
- Summaries of previous subsurface explorations and geotechnical laboratory testing
- Interpretations and characterization of subsurface conditions encountered
- Discussions of design analyses completed
- Project-specific foundation and geotechnical engineering design recommendations

1.1 Scope-of-Work

This report was prepared in accordance with Golder's revised proposal (P23-86667) for professional engineering services, dated February 22, 2012, which was amended to an existing subconsultant agreement, between M&M and Golder, dated October 21, 2010.

In general, Golder's scope-of-work in connection with the "*final design*" of the subject project involved the following professional engineering services:

- Project coordination and management
- Conduct a site-specific geotechnical subsurface exploration program[1], which was previously completed under a separate Golder proposal (P03-86509), dated August 2, 2010, for professional engineering services
- Perform geotechnical laboratory testing, as necessary, on selected, representative soil and rock samples
- Complete foundation and geotechnical engineering design analyses and evaluations, as appropriate

[1] See Golder's report, titled "*Revised Pre-Design Geotechnical Investigation Report, Norfolk Southern Railway Bridge No. 361.66 Project, Portageville, Wyoming and Livingston Counties, New York*", dated September 30, 2011, for the findings and results of this site-specific subsurface exploration program.

- Develop project-specific foundation and geotechnical engineering design recommendations, as appropriate
- Assist with the development of foundation-related construction drawings, details, and technical specifications, as appropriate
- Prepare the following site-specific reports: a) this Geotechnical Engineering Report; and b) a Geotechnical Data Report, which was previously issued on January 24, 2013 and should be incorporated into the subject project's Construction Documents

1.2 Coordinate System

Unless noted otherwise herein, the following survey controls / datums were utilized:

- **Horizontal Control:** Plan locations reference the New York State Plane Coordinate System, North America Datum of 1983 (NAD83).
- **Vertical Datum:** Elevations reference the North American Vertical Datum of 1988 (NAVD88), which is 0.522 feet below the National Geodetic Vertical Datum of 1929 (NGVD29).

Locations and ground surface elevations of borings drilled by Maxim Technologies of New York, Inc. (Maxim) in connection with the subject project (i.e., E-1 through E-3 and W-1 through W-3), as shown on Figures 2 through 6, were based on the horizontal and vertical survey coordinates shown on Drawing #2 of Maxim's report (not included herein), titled *"Preliminary Geotechnical Evaluation – Conrail Bridge No. 361.66, Letchworth State Park, Portageville, New York"*, dated March 1999.

Locations of Golder (2011) borings were established, in the field, by approximating the corresponding track stations / offsets for each boring, using hand-taped measurements and fixed survey control points installed, by others, along the proposed railway alignment. Once the track station / offset of each Golder (2011) boring was established, the corresponding horizontal coordinates (i.e., northings and eastings) and ground surface elevations were obtained using the project's designated topographic base map, as provided by M&M.

2.0 PROJECT DESCRIPTION

2.1 Project Setting and History

The subject site is located within the southwest part of Letchworth State Park (Park) in Western New York State, about forty (40) miles southwest of Rochester, New York (see Figure 1). The Park includes a scenic river gorge with three (3) waterfalls (i.e., Lower, Middle, and Upper Falls), and is commonly known as the *"Grand Canyon of the East"*. Numerous cultural and historic resources, such as the Glen Iris Inn and William Pryor Letchworth Museum, are also located in the Park.

The existing Norfolk Southern Railway Bridge No. 361.66, also known as the High Bridge, carries a single railroad track along Norfolk Southern's Southern Tier Route across the Genesee River, about 245 feet above the floor of the river gorge, within the southwest part of the Park, and about 0.75-miles northwest of the Village of Portageville in Wyoming and Livingston Counties, New York (see Figure 1). Furthermore, the existing High Bridge is situated just upstream of the Park's Upper Falls, supported on six (6) steel towers, and spans 819-feet, between its east and west abutments, across the Genesee River gorge.

The existing High Bridge replaced a pre-existing (i.e., the original) wood trestle bridge erected in 1851. The original bridge structure was owned by the Erie Railroad, was destroyed by fire in 1875. The existing iron trestle High Bridge was constructed, under an accelerated schedule, within the same year the original bridge was lost to fire (i.e., 1875). Three (3) spans of pin-connected deck trusses and ten (10) spans of deck plate girders were replaced in 1903.

Based on historic photographs, the original wood trestle bridge was founded on piers constructed from dimension stone blocks, and it appears that several of the original dimension stone foundation piers were: a) reused to support the existing bridge's steel towers; and b) resurfaced with concrete.

In the 1950s, several areas of the exposed rock slope faces, which form the gorge sidewalls, were armored with shotcrete (i.e., pneumatically applied concrete) to reduce differential weathering and erosion. In addition, it appears that dental concrete was placed within open rock joints, within the river channel, to reduce the potential for scour around the existing bridge foundation piers. However, when this dental shotcrete was applied is not known with certainty.

2.2 Site Topography and Physiography

Within the general vicinity of the project site, the Genesee River flows from south-to-north, and trends in a north-northeast (i.e., average bearing of 027-degrees) direction. The river is also deeply incised creating a 240-foot-deep, steep, narrow gorge within the vicinity of the project site. Local site conditions and topography are shown in Figure 1.

The west side of the project site is accessible from the Park's south entrance access road, which runs atop the west gorge side slope (see Figure 2), and Golder understands this access road will be closed throughout construction of the new bridge. The east side of the project site is accessible from Portageville Road and a private, unimproved access road, which runs down to the bottom of the gorge and continues along an abandoned, narrow gauge railway bed along the east bank of the Genesee River.

The bottom of the Genesee River gorge at the project site, along the centerline of the proposed new bridge alignment, is situated at about elevation (El.) +1080, while the tops of the gorge sidewalls are situated between approximately El. +1260 and El. +1265.

To the north of the existing High Bridge, the west side of the gorge slopes downward at a grade of about 50-percent until plunging nearly vertical for the lower two-thirds of the gorge. Under the existing High Bridge, a 80-foot-high, sub-vertical face exists on the west side of the gorge, and this slope face appears to be a rock outcrop, which has been covered with pneumatic shotcrete with stub drains to mitigate differential weathering, raveling, and erosion of the existing rock slope. From the existing High Bridge to the south, a wedge of talus skirts the base of the west gorge walls, resulting in a flattening of the near vertical rock slope face to an angle of repose of about 35-degrees. Slope vegetation consists predominantly of dense conifers on the upper overburden slopes, while the lower, near-vertical rock and talus slopes are sparsely vegetated with deciduous growth.

The east side of the gorge slopes more gently with grades on the order of 30- to 40-percent. A 70-foot-high, sub-vertical face exists midway down the slope of the current High Bridge alignment. This slope face appears to be a rock outcrop, which has been covered with pneumatic shotcrete with stub drains to mitigate differential weathering, raveling, and erosion of the existing rock slope. Ground surfaces on the east gorge side slopes are vegetated with dense deciduous forestation and heavy undergrowth of brush, grasses, and weeds.

See Figures 2 through 6 for plan and cross-section views of the existing site topography. See Golder's report, titled *"Report on Preliminary Geotechnical Investigation, Norfolk Southern Railway Bridge No. 361.66, Portageville, New York",* dated June 16, 2009, for additional information and representative photographs of the existing High Bridge and gorge side slope conditions.

2.3 Proposed Construction

The aging High Bridge is an important yet weak link on Norfolk Southern's Southern Tier Route, which runs between Buffalo and Binghamton, New York. Current train traffic across the existing High Bridge is moderate (i.e., about 6 to 8 train crossings per day). In addition, the existing High Bridge is the last bridge on the Southern Tier Route not capable of carrying 286,000-lbs gross weight freight cars, and speeds across the existing High Bridge are currently limited to about ten (10) miles-per-hour, due to its condition and age.

That said, the planning and design objectives[2] of the subject project were as follows:

- Eliminate operational constraints caused by the existing High Bridge
- Minimize dangerous interaction of railway activities and Letchworth State Park patrons
- Minimize dangerous and costly maintenance
- Optimize existing infrastructure and planned improvements to the Southern Tier Route

Overall, Norfolk Southern and the New York State Department of Transportation (NYSDOT) evaluated at least nine (9) different design alternatives, including replacement, rehabilitation, or abandonment of the rail route; and decommissioning of the existing High Bridge. In the end, the preferred design alternative was to construct a new railway bridge and deconstruct the existing High Bridge.

In summary, the scope of the subject project will involve the following:

- Construction of a new 963-foot-long, single-track railway bridge across the Genesee River gorge, which will be offset 75-feet south of the existing High Bridge alignment and comprise the following:
  - 483-foot-long central steel arch, supported on two (2) main reinforced concrete arch skewback foundations, situated about 235-feet above the river level.
  - 240-foot-long steel girder approach spans, on both sides of the new bridge, supported on four (4) pile-supported, reinforced concrete piers and two (2) pile-supported, reinforced concrete abutments.
- Earthwork and grading for the construction of new approach railway embankments, which will involve about 1,075- and 1,328-foot-long new approach embankments on the west and east sides of the Genesee River gorge, respectively.
- Construction of mainline railroad roadbed, private drive, and new at-grade rail crossings. Track work will be conducted between about track stations 178+84 and 221+00 (i.e., about 4,216-feet).
- Deconstruction of the existing High Bridge, which will involve the following:
  - Removal of the 819-foot-long steel viaduct consisting of treated timber, open decking, and three (3) spans of pin-connected deck trusses
  - Removal of six (6) supporting steel towers and associated twelve (12) piers
  - Removal of both the east and west bridge abutments
  - Removal of about 4,215-feet of existing single-line track work

[2] Per *"Portageville Bridge Project, Final Scoping Document",* dated March 2009, prepared by the New York State Department of Transportation and Norfolk Southern.

- Removal or adjustment of existing approach embankments

- Construction of Letchworth State Park improvements, which will involve the following:
  - Relocation / realignment of about 511.5-feet of the Park's south entrance access road
  - Construction of a new parking area along the Park's south entrance access road
  - Relocation of about 310-feet of the Gorge Trail with stone masonry railing
  - Relocation of about 474-feet of the Jemison Trail

- Erection of about 989-feet of new right-of-way fencing

Based on the length and geometry of the proposed 493-foot-long central steel arch span, the two (2) main reinforced concrete arch skewback foundations will be embedded between 20- and 85-feet behind / into the existing gorge rock slope faces (see figures 5 through 8), which will require significant rock excavation and removal (i.e., upwards of 16,000 cubic-yards for both the east and west rock cuts) to construct the proposed arch skewback foundations.

Furthermore, the bottoms of the proposed east and west arch skewback foundations will be founded El. +1149, which is about 70-feet above the normal river level and 115- to120-feet below the tops of the east and west gorge sidewalls.

See Figure 2 for the proposed new bridge alignment. See Figures 3 through 8 for the general configuration of the proposed new bridge, relative to the existing river gorge.

3.0 REGIONAL AND SITE GEOLOGY

3.1 Regional Setting

Geologically, the subject project site is located on the northern edge of the Allegheny Plateau in western New York State. The Allegheny Plateau is also situated on the northwest part of the Appalachian Plateau, which is on the west flank of the Appalachian Geologic Province.

The Appalachians were uplifted during the Ordovician through the Permian periods during two (2) separate tectonic collisions between the North American, European, and African continents, which occurred over a period from about 500 to 225 million years ago.

This region was later modified by four (4) major glacial advances / retreats from about 2 million years ago to 6,000 years ago (van Diver, 2003). The Wisconsin glaciation (peaking about 20,000 years ago) was the final glacial surge leaving the Valley Heads Moraine and a series of recessional moraines at the receding glacial front. Temporary meltwater lakes formed between the retreating ice sheet front and these moraines, establishing a base level for erosion. Rivers flowing toward these lakes down-cut into the underlying bedrock until they emptied into the lakes, where they deposited their sediment loads at the erosional base level. Lake levels dropped as the impounding moraines were overtopped and new spillways were established at lower elevations, creating new lower base levels which reactivated erosion and down-cutting (van Diver, 2003) to produce the series of waterfalls (Upper, Middle, and Lower Falls) downstream of the project site.

In general, exposed bedrock within the region belongs to the Late Devonian Nunda Formation of the West Falls Group, which is about 400- to 950-feet-thick and characterized by fine-grained sandstones with interbedded shale layers (Rickard and Fisher, 1970; Clarke and Luther, 1908). Furthermore, the Nunda Formation is most likely a submarine fan deposit, as its sandstones are generally thick, massive to wavy / flaggy-bedded, have few primary sedimentary structures, terminate abruptly, and appear to have lobate forms (Jacobi et. al., 1994).

3.2 Site Geology

At the subject project site, geologic conditions consist of overburden soils overlying sub-horizontally-bedded sedimentary bedrock. Overburden soils generally consist of glacial drift (till and outwash) and colluvium. The underlying bedrock consists of fine-grained silty sandstones with interbedded shale layers of the Nunda Formation member of the West Falls Group (Rickard and Fisher, 1970; Clarke and Luther, 1908).

In general, interbedded shale layers within bedrock appear to increase in thickness and frequency within the lower portion of the Genesee River gorge sidewalls. Inversely, sandstone layer thicknesses appear to decrease vertically downward (i.e., sandstone thicknesses are thinner in the lower portion of the gorge sidewalls, and are thicker in the upper portions of the gorge sidewalls).

Between El. +1145 and +1155, an approximate 6- to 9-foot-thick layer of shale and/or shaly-sandstone lies at / near the bases of the proposed east and west arch skewback foundations (see Figures 5 and 6). Furthermore, this shale / shaly-sandstone layer appears to coincide with a *"hard blue shale"* identified in Clarke and Luther (1908), and was also observed, identified in the Golder (2008) field geologic exploration program.

4.0 PREVIOUS SUBSURFACE EXPLORATIONS

To date, three (3) separate, independent subsurface explorations were undertaken and completed in connection with the subject project, which are generally described and summarized below.

4.1 Maxim (1999) Preliminary Subsurface Exploration Program

Between January and February 1999, Maxim conducted a subsurface exploration[3] consisting of six (6) borings (i.e., borings E-1, E-2, and E-3 and W-1, W-2, and W-3 drilled on the east and west sides of the river, respectively) to depths ranging between 120.0 and 140.5 feet below ground surface (bgs).

See Figures 2 thru 6 for the Maxim (1999) borehole locations and drilled depths. See Appendix A for copies of Maxim (1999) boring and rock core photo logs.

See Maxim's report, titled *"Preliminary Geotechnical Evaluation – Conrail Bridge No. 361.66, Letchworth State Park, Portageville, New York"*, dated March 1999, for additional information regarding its preliminary geotechnical evaluation of the subject project.

4.2 Golder (2008) Field Geologic Exploration Program

This field geologic exploration program did not include the drilling of new, additional borings, but did include the following field exploration activities:

- **Geologic Rock Mapping:** In June 2008, field geologic mapping, in accordance with International Society of Rock Mechanics (ISRM) protocols[4], was conducted on the gorge side slope rock faces and outcrops within the vicinity of the existing High Bridge and proposed bridge alignments. Data collection included lithologic descriptions of rock types and measurement of discontinuities (e.g., joints, fractures, and bedding) observed within the exposed rock faces.

  Rock slope geometry, lithology (e.g., color and grain size), and rock mass information (e.g., number and spacing of joint sets) were also noted. In general, collected rock discontinuity data included, but was not necessarily limited to, joint types, dip directions and angles, joint persistence and termination, joint spacing, aperture widths, nature and strength of joint infilling, and roughness and shape of joint surfaces.

  Mapped rock faces were accessed either on foot or by fixed-line rappelling techniques, as required and appropriate to obtain the necessary measurements. In total, 270 measurements were collected within the following three (3) project sections / zones: 1) northwest gorge side; 2) southwest gorge side; and 3) east gorge side.

- **Geophysical Surveys:** In June 2008, ground penetrating radar (GPR) surveys were performed to evaluate and assess the conditions of the existing High Bridge's shotcrete-coated foundation piers and shotcrete-faced rock slopes within the vicinity of the existing High Bridge. These surveys were also carried out using fixed-line rappelling techniques.

[3] All soil and rock core samples collected from the Maxim (1999) borings were not available for Golder's inspection, classification, or testing, and the current status and location of these soil and rock core samples are not known.
[4] International Society of Rock Mechanics (1981) Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses

In particular, the objectives of these GPR surveys were to: a) investigate the presence of voids behind the shotcrete facings; and b) estimate applied shotcrete thicknesses.

See Golder's report, titled *"Report on Preliminary Geotechnical Investigation, Norfolk Southern Railway Bridge No. 361.66, Portageville, New York",* dated June 16, 2009, for additional information regarding its geologic field exploration program, as described above.

4.3 Golder (2011) Pre-Design Subsurface Exploration Program

Between January and March 2011, a total of seventeen (17) borings (i.e., 4 deep, inclined borings and 13 shallow, vertical borings) were drilled along the proposed railway alignment to depths ranging between 18- and 170-feet bgs. In addition, four (4) piezometers were constructed within or adjacent to selected shallow vertical borings.

See Figures 2 thru 6 for the Golder (2011) borehole locations and drilled depths. See Appendices B and C for copies of Golder (2011) boring and piezometer logs and rock core photo logs, respectively.

In summary, the Golder (2011) pre-design subsurface exploration program included the following:

- **IB-Series Borings:** Four (4) deep, inclined rock core boreholes (i.e., IB-1 through IB-4) were advanced through the overburden deposits, without collecting split-spoon samples, utilizing Hollow Stem Auger (HSA) drilling techniques. Borehole inclinations ranged between five (5) and fifteen (15) degrees off the vertical axis of each borehole.

  Once bedrock was encountered, permanent 4-inch-diameter steel casings were installed and grouted into bedrock. These borings were further advanced utilizing NQ-sized, wire-line rock coring methods, and rock cores were typically collected in 10-foot lengths (i.e., core runs) to inclined lengths/depths ranging between 169- and 170-feet.

  Collected rock cores were placed within wooden core boxes. In addition, collected rock core was logged in the field, and rock core recovery and rock quality designation (RQD) values were calculated and recorded on the boring logs. Upon completion, these inclined boreholes were capped and locked.

- **B-Series Borings:** Thirteen (13) shallow, vertical borings (i.e., B-1 through B-13) were advanced through the overburden deposits, utilizing hollow stem auger (HSA) drilling techniques, with depths ranging between 18- and 88-feet bgs.

  Between the as-drilled ground surface and about 35-feet bgs, split-spoon soil samples were collected continuously, utilizing standard 2-inch-diameter samplers, and thereafter were collected on 5-foot intervals to bedrock. Standard Penetration Tests (SPT) were also performed, in accordance with ASTM D-1586, and the corresponding SPT N-values (i.e., blow counts) were recorded on the boring logs.

  Collected split spoon soil samples were visually classified in the field, in general accordance with the Unified Soil Classification System (USCS), and appropriate soil descriptions were recorded on the boring logs. In addition, representative soil samples were preserved for future inspection, classification, and geotechnical laboratory testing.

Bedrock was cored within nine (9) of the thirteen (13) B-Series borings (i.e., B-4 through B-12), utilizing NQ-sized rock coring methods. In borings where rock was cored, the collected rock cores were placed within wooden core boxes, logged by Golder, and rock core recovery and rock quality designation (RQD) values were calculated and recorded on the boring logs.

Upon completion of drilling activities, each borehole not converted into a piezometer was infilled with drill cuttings, and sealed with cement-bentonite grout to the ground surface. In borings where rock coring was performed, said boreholes were infilled with cement-bentonite grout from bottom to top-of-rock, and thereafter infilled with drill cuttings and cement-bentonite grout, as necessary, to the ground surface.

- **Piezometers:** Four (4) 2-inch-diameter, polyvinyl chloride (PVC) standpipe piezometers were constructed within borings B-3, B-7, B-8, and B-12 with screen lengths varying between 10- and 20-feet. For PZ-3, PZ-8, and PZ-12, protective steel standpipe riser casings were installed above the ground surface. For PZ-7, a flush-mounted road box was installed, due to its proximity to the adjacent Park access road.

- **Geophysical Surveys:** Given Golder's drilling subcontractor (i.e., Nothnagle Drilling Inc. of Scottsville, New York) did not have nor could procure an orientated rock core barrel, structural rock discontinuity measurements (e.g., dip angle and direction) could not be obtained directly from the extracted rock core. Therefore, Golder elected to log each of the accessible Golder (2011) IB-Series and Maxim (1999) boreholes, utilizing the following borehole geophysical logging equipment and methods:

  - **Caliper Probe:** A caliper probe was used to measure changes in borehole diameter, and these data were used to indicate locations where rock mass discontinuities (e.g., joints and fractures) intersect the borehole sidewall.
  - **Optical Televiewer (OTV):** An OTV instrument was used to provide high-resolution, continuous, 360-degree unwrapped images of the in-situ borehole sidewalls, utilizing a DSP based digital CCD camera, a conical mirror, and a LED light source. Borehole images generated by the OTV were orientated using data recorded from a 3-axis magnetometer and multiple accelerometers incorporated into the OTV.
  - **Acoustic Televiewer (ATV):** An ATV instrument is similar to an OTV instrument, but instead uses ultrasonic beams, reflected off borehole sidewalls, to create 3-D acoustic caliper maps of the borehole sidewalls. Borehole images generated by the ATV were orientated using data recorded from a 3-axis magnetometer and multiple accelerometers incorporated into the ATV.

  Between March 28 and 30, 2011, the above-noted borehole geophysical equipment and methods were used to create, collect high-resolution borehole images, and these data were subsequently used to identify and compute associated structural geologic data for boreholes IB-1, IB-2, IB-3, E-1, E-2, E-3, and W-3. In total, 631 discontinuity measurements were obtained from the OTV/ATV data.

  See Appendix D for a copy of Golder's report, titled *"Borehole Geophysical Investigation Report"*, dated June 2011, for additional information related to above-noted borehole geophysical surveys.

Soil and rock core samples collected from the IB- and B-Series borings are currently stored and accessible within Norfolk Southern's Hornell, New York storage / maintenance facility.

See Golder's report, titled "*Revised Pre-Design Geotechnical Investigation Report, Norfolk Southern Railway Bridge No. 361.66 Project, Portageville, Wyoming and Livingston Counties, New York*", dated September 30, 2011, for additional information regarding Golder's pre-design subsurface exploration program, as described above.

5.0 GEOTECHNICAL LABORATORY TESTING

5.1 Soil Sample Testing

5.1.1 Golder (2011) Pre-Design Subsurface Exploration Program

As part of Golder's pre-design geotechnical subsurface exploration program, as described above, collected soil samples were transported to Golder's Newark, New Jersey office for further examination and visual inspection/classification. In addition, representative "*disturbed*" soil samples were transported to TerraSense, LLC of Totowa, New Jersey for testing, which included the following:

Soil Sample Test	ASTM Reference	No. Tests
Moisture Content	D-2216	62
Organic Content	D-2974	8
Atterberg Limits	D-4318	21
Sieve Analyses	D-422	37
Sieve / Hydrometer Analysis	D-422	21

See Appendix E for copies of the above-noted soil testing results.

5.2 Rock Core Testing

5.2.1 Maxim (1999) Preliminary Subsurface Exploration Program

As part of its preliminary geotechnical evaluation, Maxim[5] selected representative rock core samples for testing, which included the following:

Rock Core Test	ASTM Reference	No. Tests
Unconfined Compressive Strength	D-2938[6]	12
Elastic Moduli	D-3148[5]	4

See Appendix F for copies of the above-noted rock core testing.

5.2.2 Golder (2008) Field Geologic Exploration Program

As part of its field geologic exploration program, Golder collected representative rock block samples, which were shipped to the Earth Mechanics Institute at the Colorado School of Mines in Golden, Colorado. In addition, these rock block samples were cored, in the laboratory, both parallel and perpendicular to bedding, and the following tests were performed of these cored rock samples:

[5] The Maxim (1999) rock core was not available for Golder's inspection or testing, and the current status and location of this rock core is not known.
[6] ASTM D-2938 and D-3148 were withdrawn from circulation in 2005, and replaced by ASTM D-7012.

Rock Core Test	ASTM Reference	No. Tests
Compressive Strength	D-7012	5

See Appendix F for copies of the above-noted rock core testing.

5.2.3 Golder (2011) Pre-Design Subsurface Exploration Program

Upon completion of field activities associated with Golder's pre-design subsurface exploration program, collected rock core boxes were transported to and are currently accessible within Norfolk Southern's Hornell, New York storage facility. In addition, representative rock core samples were selected and transported to TerraSense, LLC of Totowa, New Jersey for testing, which included the following:

Rock Core Test	ASTM Reference	No. Tests
Compressive Strength	D-7012	20
Elastic Moduli	D-7012	20
Direct Shear Strength	D-5607	4

See Appendix F for copies of the above-noted rock core testing.

5.3 Pyrite Testing

Upon further inspection of the collected Golder (2011) rock core, Golder selected eight (8) representative rock cores, which can be generally characterized as sandstone / siltstone, shaly-sandstone, or shale, and shipped said rock core samples to its rock testing laboratory in Burnaby, British Columbia, Canada for additional pyrite testing. The following tests were performed on each rock core sample:

- Petrographic (i.e., thin section) examinations
- X-ray diffraction (XRD) analyses

See Appendix G for copies of the above-noted pyrite testing results.

6.0 INTERPRETED SUBSURFACE CONDITIONS

In general, geologic conditions at the subject project site consist of overburden soils overlying sub-horizontally-bedded, grey, fine-grained sandstones / siltstones with interbedded shale layers of the Nunda Formation member of the West Falls Group. Bedrock was encountered and cored in all six (6) Maxim (1999) boreholes and 13 of 17 Golder (2011) boreholes, and top-of-rock was found to vary between about 13- and 78-feet bgs.

See Figure 2 for borehole locations. See Figures 3 through 6 for interpreted subsurface profiles along the proposed railway alignment.

Subsequent sections herein provide additional information, descriptions, and characterizations of the interpreted overburden, bedrock, and groundwater conditions encountered at the subject project site.

6.1 Overburden

In general, overburden conditions can be described and characterized, in descending geologic order (i.e., from youngest to oldest or from the ground surface vertically downward), as follows:

- **Fill and Topsoil (Stratum 1):** This stratum was encountered in all borings, and was found to be 2- to 12-feet-thick. In general, this stratum comprises heterogeneous mixtures of black, brown, and grey, fine-to-coarse sand with varying amounts of gravel, silt, clay, crushed stone, and deleterious components (e.g., topsoil, roots, wood, concrete, brick, and HDPE). These fill materials may be associated with the existing railway embankment construction or maintenance activities adjacent to the south side of the existing railway embankments.

  SPT N-values measured within this stratum ranged from two (2) to thirteen (13) blows-per-foot (bpf) and averaged seven (7) bpf. This stratum can be generally classified as USCS Class SP, SM, SC, GP, GM, and GC.

- ***Silt, Sand, and Gravel (Stratum 2):*** This stratum was encountered in all borings, and was found to be 9- to 37-feet-thick. In general, this stratum comprises brown and grey; fine-to-coarse sand with trace-to-some silt, clay, and gravel; silty, clayey, gravely sand; sandy, silty, clayey gravel; gravel with trace-to-some sand, silt, and clay; silt; sandy silt; and sandy silt and clay. This stratum also contains intermittent (i.e., non-spatially present) thin clay layers or lenses.

  SPT N-values measured within this stratum ranged from two (2) to sixty-three (63) bpf and averaged nineteen (19) bpf, and these blow counts appear to increase with depth. This stratum can be generally classified as USCS Class SP, SM, SC, SC-SM, SM-ML, ML, GP, GM, GC-GM, and GP-GM with occasional interbedded layers or lenses of CL.

- ***Clay (Stratum 3):*** This stratum was encountered within eight (8) of thirteen (13) B-series borings (i.e., B-1, B-2, and B8 through B-13), and was found to be 5- to 60-feet-thick[7]. In general, this stratum comprises grey and brown; clay with trace-to-little fine-to-coarse sand and trace fine gravel; silty, sandy clay; silt; and sandy silt. This stratum may be

[7] Weathered bedrock and/or bedrock was not encountered within borings B-1 and B-2. Hence, actual thicknesses of Stratum 3, within the western portion of the project site, are not known with certainty.

representative of a glaciolacustrine clay deposit, which can exist within the vicinity of the project site.

SPT N-values measured within this stratum ranged from eleven (11) to seventy-four (74) bpf and averaged thirty (30) bpf. This stratum can be generally classified as USCS Class CL, CL-ML, and ML.

- ***Weathered Rock:*** This stratum was encountered within eleven (11) of thirteen (13) B-series borings (i.e., B-3 through B-13), and was found to be 2- to 15-feet-thick[8]. In general, this stratum consists of heterogeneous mixtures of silt, clay, sand, gravel, and rock fragments derived from grey, highly weathered to weathered sandstone, siltston, and shale materials. In addition, the degree of weathering decreases with depth (i.e., transitions from highly weathered to weathered to fractured bedrock).

  SPT N-values measured within this stratum ranged from thirty-two (32) bpf to one-hundred (100) blows-per-one-inch-penetration (i.e., split-spoon refusal). Excluding all split-spoon refusal values, N-values within this stratum varied between thirty-two (32) and one-hundred-twenty-three (123) bpf and averaged sixty (60) bpf. In addition, this stratum can be generally classified as USCS Class GP, GM, GC, GP-GM, SP-GM, and CL-GP.

6.2 Bedrock

In general, the underlying rock mass (i.e., bedrock) can be described and characterized as follows:

- The underlying rock mass consists of fair-to-good quality, sub-horizontally-bedded, grey, fine-grained sandstone, siltstone, and silty-sandstone interbedded with thin, typically 2- to 8-inch-thick, shale layers. These thin shale layers (i.e., interbeds) account for about one (1) to three (3) percent of the as-drilled rock mass.

- Between El. +1145 and El. +1155, an approximate 6- to 9-foot-thick layer of shale and/or shaly-sandstone, as shown on Figures 5 and 6, exists at or near the bases of the proposed east and west arch skewback foundations. Furthermore, this shale / shaly-sandstone layer appears to coincide with the *"hard blue shale"* identified in Clarke and Luther (1908), and was also observed in the Golder (2008) field geologic exploration program.

- Based on the borehole data, RQD values range between 0% and 100% (i.e., very poor to excellent rock quality), and average 78% (i.e., good quality rock). Within the vicinity of the proposed arch skewback foundations, between El. +1100 and El. +1200, RQD values range between 47% and 100% (i.e., poor to excellent rock quality), and average 86% (i.e., good quality rock).

- The table below further provides a summary of RQD values, between El. +1100 and El. +1200, within the east, west, and combined gorge side slopes.

Elevation (feet)		Rock Quality Designation (RQD), %								
		East & West Sides			East Side			West Side		
from	to	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.
1175	1200	47.0	100.0	88.6	47.0	100.0	86.8	68.0	100.0	90.5
1150	1175	49.0	100.0	84.9	49.0	100.0	83.1	69.0	100.0	86.4

[8] Weathered bedrock thicknesses may vary widely across the project site, due to uncertainties with respect of the degree of weathering of the underlying bedrock formation.

Elevation (feet)		Rock Quality Designation (RQD), %								
		East & West Sides			East Side			West Side		
from	to	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.
1125	1150	68.3	100.0	88.9	68.3	100.0	86.7	80.0	98.0	91.8
1100	1125	53.3	99.2	82.4	53.3	100.0	79.6	77.5	99.2	89.9

- Based on the combined geologic mapping and OTV/ATV discontinuity data (i.e., 901 measurements), the table below contains a summary of the structural discontinuity (e.g., joint) sets identified within the underlying rock mass:

Joint Set	Label	Type	Avg. Dip Angle (degrees)	Avg. Dip Direction (degrees)
Bedding	B1	Bedding	01	058
Joint 1	J1	Major	83	148
Joint 2	J2	Major	84	067
Joint 3	J3	Minor	60	130
Joint 4	J4	Minor	79	105
Joint 5	J5	Minor	54	097
Joint 6	J6	Minor	54	037
Joint 7	J7	Minor	74	320
Joint 8	J8	Minor	89	031

See Appendix H for associated rock discontinuity stereonet plots[9].

- Bedding discontinuities can be generally characterized as follows: a) have very low (less than 3-feet) to high (30- to 60-feet) persistence; b) are spaced 0.1- to 10-feet; c) have very tight (less than 0.1-millimeter) to cavernous (greater than 1-meter) openings; d) are planar to stepped to undulating in shape; e) are rough to smooth; and f) open bedding discontinuities contain varying degrees of infilling, consisting of silty sand, shale, clay, broken rock, and rare secondary mineralization (e.g., chlorite, talc, and gypsum).

- Joint discontinuities can be generally characterized as follows: a) have very low (less than 3-feet) to very high (greater than 60-feet) persistence; b) are spaced 0.1- to 25-feet; c) have very tight (less than 0.1-millimeter) to extremely wide (10- to 100-centimeter) openings; d) are planar, irregular, curved, and stepped in shape; e) are smooth to very rough, and rarely polished and/or slickensided; and f) open joint discontinuities contain varying degrees of infilling, consisting of silty sand, clay, broken rock, and rare secondary mineralization (e.g., chlorite, talc, gypsum, and iron oxide).

- The OTV/ATV discontinuity data indicate bedding spacing ranges from 0.01- and 16.5-feet, and bedding apertures range from very tight (less than 0.1-millimeter) to very wide (1- to 10-centimeter). In addition, the OTV/ATV discontinuity data indicate joint spacing ranges from 0.02- to 25-feet with apertures ranging from very tight (less than 0.1-millimeter) to very wide (1- to 10-centimeter).

[9] Rock discontinuity stereonet plots were generated using the *Dips* software (Ver. 5.109), as developed and distributed by Rocscience, Inc. of Toronto, Ontario, Canada.

- Based on the Maxim (1999), Golder (2008), and Golder (2011) rock strength testing results, the rock mass has the following characteristics:
  - Rock unit weights range between 151.0- and 166.0-pcf[10], and average 157.6-pcf
  - Intact rock uniaxial compressive strengths (UCS_i) range between 5,940- and 18,800-psi[11], and average 14,812-psi
  - Intact rock elastic moduli (E_i) range between 880,000- and 4,000,000-psi, and average 2,705,417-psi
  - Intact rock modulus ratios (MR_i) range between 134 and 240, and average 178
  - Poison's ratios (μ_r) range between 0.04 and 0.19, and average 0.11
- Based on the Golder (2011) direct shear rock testing results, bedding discontinuities have the following strength characteristics:
  - Effective (peak) friction angles (ϕ'_{peak}) range between 24.7- and 35.74-degrees, and average 29.4-degrees
  - Effective (final) friction angles (ϕ'_{final}) range between 22.2- and 36.4-degrees, and average 29.0-degrees
  - Peak cohesion (c_{peak}) values range between 3.1- and 19.5-psi, and average 9.1-psi
  - Final cohesion (c_{final}) values range between 0.0- and 22.0-psi, and average 6.5-psi
- Based on the Golder (2011) pyrite testing results, the underlying sandstone / siltstone and interbedded shale / shaly-sandstone layers have the following pyrite concentrations:
  - **Sandstone / Siltstone:** less than 1% pyrite
  - **Shale / Shaly Sandstone:** less than 1% to 3% pyrite

6.3 Groundwater

At the end of the Golder (2011) subsurface exploration field activities, the following groundwater levels were measured within the installed shallow piezometers (i.e., PZ-3, PZ-7, PZ-8, and PZ-12) and inclined boreholes (i.e., IB-1 through IB-4):

- **West Side of Gorge:** Shallow, perched groundwater within the overburden materials was not encountered within piezometers PZ-3 and PZ-7. Within IB-1 and IB-2, groundwater was measured between El. +1110 and El. +1130.
- **East Side of Gorge:** Shallow, perched groundwater was measured at El. +1254.5 and El. +1303.3 within PZ-08 and PZ-12, respectively. Within IB-3 and IB-4, groundwater was measured between El. +1130 and El. +1150.

[10] pcf = pounds-per-cubic-foot
[11] psi = pounds-per-square-inch

7.0 DISCUSSION

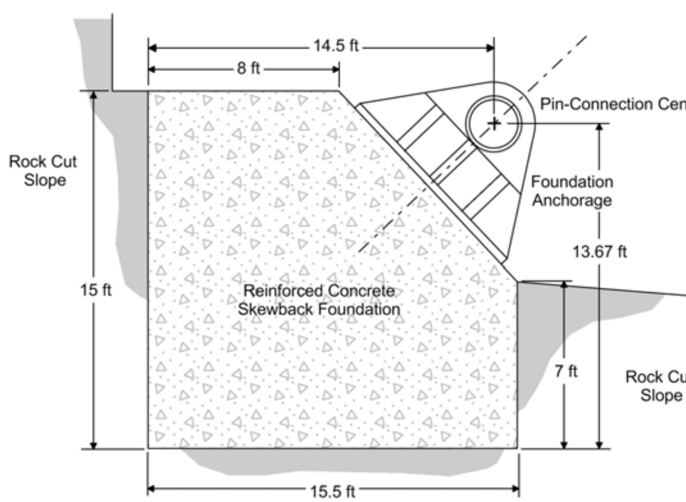
7.1 Arch Skewback Foundation Design

In summary, the following generally accepted approach was implemented to: a) establish reasonable rock mass properties; b) compute bearing capacities for foundations on rock; and c) estimate magnitudes of foundation deformations:

- Perform geologic field reconnaissance to collect structural geology data from exposed rock out-crops and slopes (see Section 4.2 herein).
- Drill a series of boreholes to collect rock core samples. Inspect, describe, and log the collected rock core (see Appendices A and B). Compute core recoveries and RQD values for each rock core run (see Section 6.2 herein). Perform borehole geophysical surveys (see Section 4.3 herein) to collect structural geology data from these boreholes.
- Establish the number, orientation, and characteristics of identified rock mass discontinuity (e.g., joints, fractures, and bedding) sets, based on the borehole and geologic rock mapping data (see Appendix H).
- Select representative intact rock core samples for uniaxial compressive strength (UCS), elastic moduli, and direct shear strength testing (see Appendix F).
- Establish representative RQD, intact rock strength, and intact rock elastic moduli values and compute rock mass ratings (RMR)[12] for each delineated unit of jointed rock (see Appendix J).
- Use established empirical relationships (e.g., Hoek-Brown failure criterion[13]) to scale intact rock properties to establish reasonable rock mass properties (see Section 7.1.4 herein).
- Compute rock mass bearing capacities (see Section 7.1.5 herein).
- Estimate foundation deformations (see Section 7.1.6 herein).

7.1.1 Design Section

In general, the geometry and dimensions of the proposed arch skewback foundation are shown below. See Figures 7 and 8 for the configurations of the west and east arch skewback foundations, respectively.



[12] Bieniawski (1989)
[13] Hoek and Brown (1980, 1988), Hoek (1994), Hoek et al. (2002), and Hoek and Diedrichs (2006)

7.1.2 Structural Performance Criteria

As stipulated by M&M and/or AREMA (2011), the project-specific structural performance criteria for the proposed arch skewback foundation design are as follows:

- Maximum incremental[14] horizontal pin-connection movement = 0.1-inch
- Maximum total[15] vertical pin-connection movement = 0.5-inch
- Bearing capacity factor-of-safety (primary loads) = 3.0
- Bearing capacity factor-of-safety (primary plus secondary loads) = 2.0

7.1.3 Foundation Design Loads

As provided by M&M, the table below provides a summary of the total "*primary*"[16] (P), "*primary plus secondary*"[17] (P + S), and *"dead load at arch closure"* (DL_c) applied foundation loads on the proposed bridge anchorage pins, and the load combination / case generating the maximum total applied foundation loads is highlighted below in "**Red**". See Appendix I for a table showing all arch skewback foundation design load combinations, as provided by M&M.

	Applied Foundation Loads[18] on Pin-Connection						
	Dead Load at Arch Closure	(P) Loads				(P + S) Loads	
Arch Skewback Foundation Load Case =	DL_c	1	2	3	4	1	2
F_X, kips =	2,400	5,976	5,732	6,336	6,092	**9,758**	724
F_Z, kips =	3,200	7,960	7,960	8,426	8,426	**13,788**	468
F_Y, kips =	0	0	0	0	0	**550**	550
R_{X-Z}, kips =	4,000	9,954	9,809	10,542	10,398	**16,892**	862
R_{X-Z} Inclination Angle[19], degrees =	10.0	10.0	11.1	9.9	11.0	**11.6**	(10.3)

Where,

F_X = Horizontal Load on Anchorage Pins, kips[20,21]
F_Z = Vertical Load on Anchorage Pins, kips[22]
F_Y = Transverse Load on Anchorage Pins, kips[23]
R_{X-Z} = Horizontal-Vertical Resultant Load on Anchorage Pins, kips

[14] Incremental pin-connection movements were computed between the applied DL_c and maximum (i.e., primary plus secondary) loads on the proposed bridge anchorage pins.
[15] Total pin-connection movements were computed based on the maximum (i.e., primary plus secondary) applied foundation loads on the proposed bridge anchorage pins.
[16] Primary loads include all dead loads, live loads, and 50% of temperature loads.
[17] Primary plus Secondary loads include all dead, live, wind, temperature, tracking, braking, equipment, and stability live loads.
[18] Foundation design loads, as presented above, exclude the self-weight of the proposed arch skewback reinforced concrete foundations.
[19] Inclination angles, as presented above, are measured above the plane perpendicular to the bridge anchorage bearing surface and through the center of the bridge anchorage pin.
[20] 1-kip = 1000-lbs = 0.5-ton
[21] Positive F_X values are directed towards the adjacent bridge abutments (i.e., towards the rock slope).
[22] Positive F_Z values are directed vertically downward.
[23] Positive F_Y values are transverse, perpendicular to F_X and F_Z, and directed from south to north.

To further define the magnitudes and distributions of applied foundation contact stresses along the arch skewback foundation back- and bottom-bearing surfaces, a two-dimensional finite-element program (i.e., Phase[2]), as described in Section 7.1.6 herein, was utilized, and the following ranges of applied foundation contact stresses[24] were computed:

- **Back-Bearing Surface:** 9.6- to 23.6-ksf[25], and averaged 11.8-ksf
- **Bottom-Bearing Surface:** 9.0- to 28.1-ksf, and averaged 14.5-ksf

7.1.4 Rock Mass Parameters

See below for a table showing rock mass properties for two (2) conservative (i.e., lower bound) design conditions / cases (i.e., Cases 1 and 2), which were incorporated into the proposed arch skewback foundation bearing capacity calculations and numerical modeling analyses.

Property	Symbol	Design Values		Units	Comments / Typical Values
		Case 1	Case 2		
Unit Weight	γ_r	155	155	pcf	Lab: 151- to 166-pcf and avg. 159-pcf
Poisson's Ratio	μ_r	0.10	0.10	-	Lab: 0.04 to 0.19, and avg. 0.11
Rock Mass Ratings	RMR	35	49	-	
	RMR_{89}	58	64	-	
Geological Strength Index	GSI	53	59	-	
UCS of Intact Rock	σ_{ci}	8	12	ksi[26]	Lab: 5.9- to 18.8-ksi, and avg. 14.8-ksi
Modulus Ratio	MR	200	200	-	Lab: 134 to 240, and avg. 178 Sandstone: 275 ± 75 Shale: 200 ± 50
Modulus of Intact Rock	E_i	1,600	2,400	ksi	Lab: 880- to 4,000-ksi and avg. 2,700-ksi
Disturbance Factor	D	0.5	0.5	-	
Hoek-Brown Criterion Values	m_i	15	15	-	Sandstone: 17 ± 4 Shale: 6 ± 2
	m_b	1.600	2.129	-	
	s	0.0019	0.0042	-	
	α	0.505	0.503	-	
UCS of Rock Mass	σ_c	0.339	0.767	ksi	

[24] Contact stresses are perpendicular to the concrete-rock bearing surface interfaces.
[25] 1-kip-per-square-foot (ksf) = 0.5-ton-per-square-foot (tsf)
[26] 1-kip-per-square-inch (ksi) = 1,000-psi

Property	Symbol	Design Values		Units	Comments / Typical Values
		Case 1	Case 2		
Tensile Strength	σ_t	-0.009	-0.024	ksi	
Global Rock Mass Strength	σ'_{cm}	1.347	2.373	ksi	
Deformation Modulus	E_{rm}	285.4	616.6	ksi	
Confining Stress Limit	σ'_{3max}	2.0	3.0	ksi	
Equivalent Mohr-Coulomb Strength Parameters	ϕ'	30.2	32.5	degrees	
	c'	0.388	0.650	ksi	

7.1.5 Bearing Capacity

In general, rock mass ultimate bearing capacities were computed for the above-noted two (2) conservative (i.e., lower bound) design conditions / cases (i.e., Cases 1 and 2) utilizing the following five (5) computational methods:

- **Method 1:** Hoek-Brown Criterion – Lower Bound[27]
- **Method 2:** Hoek Brown Criterion – Recessed Footing[28]
- **Method 3:** Plain Strain Variables[29]
- **Method 4:** Mohr-Coulomb – General Shear Failure
- **Method 5:** Mohr-Coulomb – Compressive Failure

In summary, the computed rock mass ultimate bearing capacities (Q_{ult}) for each of the above-noted computation methods and for two (2) different foundation embedment conditions are presented below.

Method	Depth of Embedment (feet)	Q_{ult} (ksf)	
		Case 1	Case 2
1	0	400	874
	7	473	1,032
2	0	411	887
	7	578	1,137

[27] Bell (1992) and Kulhawy and Carter (1992)
[28] Wyllie (1999)
[29] Serrano et al. (1994, 2000, and 2001)

Method	Depth of Embedment (feet)	Q_{ult} (ksf)	
		Case 1	Case 2
3	0	1,208	2,547
	7	1,548	3,100
4	0	1,966	4,059
	7	2,350	4,822
5	0	194	342
	7	194	342

Assuming a maximum allowable bearing capacity equal to 40-ksf (i.e., 20-tsf), which is 11.9- and 25.5-ksf greater than the maximum and average applied foundation contact stresses, respectively, on the proposed arch skewback foundation bottom-bearing surfaces (see Section 7.1.3 herein), the computed factors-of-safety against bearing capacity failure ($FS_{bearing}$) for each of the above-noted computational methods are as follows:

- **Method 1:** $FS_{bearing}$ = 10.0 to 25.8
- **Method 2:** $FS_{bearing}$ = 10.2 to 28.4
- **Method 3:** $FS_{bearing}$ = 30.2 to 77.5
- **Method 4:** $FS_{bearing}$ = 49.1 to 120
- **Method 5:** $FS_{bearing}$ = 4.8 to 8.6

Hence, the project-specific bearing capacity structural performance criteria, as presented in Section 7.1.2 herein, were achieved.

See Appendix J for a copy of Golder's bearing capacity calculation package, titled "*Arch Skewback Foundations – Rock Mass Bearing Capacity, Norfolk Southern Bridge (361.66) Replacement Project, Portageville, Wyoming and Livingston Counties, New York*", dated October 15, 2012, for additional details and information relative to the above-noted bearing capacity computational methods.

7.1.6 Numerical Modeling

Numerical modeling was performed using the two-dimensional, plane-strain finite-element program *Phase*^{2} (Ver. 8.0; Rocscience, 2012b). *Phase*^{2} simulates the behavior of soil and rock materials by representing geologic materials as an arrangement of elements and nodes that form a grid (or mesh) that can be adjusted by the user to fit the shape of the structure being analyzed.

For the subject project, *Phase²* was used to make predictions (i.e., estimates) of the proposed arch skewback foundation deformations / displacements and contact stresses induced by the applied intermediate[30] and final (i.e., P + S) foundation loading conditions, as defined in Appendix K herein.

In total, seven (7) separate, distinct rounds of numerical modeling analyses were performed. Rounds 1, 2, and 3 of these numerical analyses were performed on the original "*trapezoidal-shaped*" arch skewback foundation design concept, which was subsequently re-designed and is not discussed further herein. Rounds 4A, 4B, 5, and 6 were performed on the current "*rectilinear-shaped*" arch skewback foundation design concept, as presented in Section 7.1.1 herein.

Based on the *Phase²* Rounds 4A, 4B, 5, and 6 numerical modeling analysis results, the proposed arch skewback foundations were estimated to have the following incremental horizontal and total vertical pin-connection movements, as defined in Section 7.1.2 herein:

- **Incremental Horizontal Movements:** ranged between 0.01- and 0.10-inch
- **Total Vertical Movements:** ranged between 0.09- and 0.27-inch

Hence, the project-specific pin-connection movement structural performance criteria, as presented in Section 7.1.2 herein, were achieved.

See Appendix K for a copy of Golder's design memorandum, titled "*Arch Skewback Foundations – Numerical Modeling Summary, Norfolk Southern High Bridge (SR-361.66) Replacement Project, Portageville, Wyoming and Livingston Counties, New York*, dated January 18, 2013, for additional details and information in connection to the above-noted numerical modeling analyses.

7.2 Piers and Abutment Foundation Design

In general, axial design capacities were computed following methods described in AASHTO (2012), and lateral design capacities were computed using the LPILE (Ver. 2012.6.34) and GROUP (Ver. 8.0.15) computer software, as developed and distributed by Ensoft, Inc. (Ensoft) of Austin, Texas.

7.2.1 Structural Performance Criteria

In summary, the deep foundations (i.e., micropiles) supporting the proposed bridge abutment and approach span piers were designed in accordance with the following project-specific structural performance criteria:

- Maximum micropile settlement = 0.5-inch
- Axial / lateral capacity design factor-of-safety = 2.5

[30] The intermediate loading condition is between the applied DL_c and maximum "*P + S*" loads on the bridge anchorage pins.

7.2.2 Foundation Design Loads

As provided by M&M[31], the table below provides a summary of the applied foundation loads on the proposed approach span pier.

	Approach Span Piers Foundation Design Loads	
Abutment and Pier Load Case =	1	2
F_P, kips =	4,103	3,804
F_T, kips =	93	93
M_T, kip-feet =	4,675	4,675
F_L, kips =	76	300
M_L, kip-feet =	3,363	15,214

Where,

F_P = Axial load on pile cap, kips
F_T = Lateral load on pile cap long-axis, kips
F_L = Lateral load on pile cap short-axis, kips
M_T = Moment about pile cap long-axis, kip-feet
M_L = Moment about pile cap short-axis, kip-feet

As stipulated by M&M, the maximum applied foundation design loads on the specified micropiles, supporting the proposed bridge abutments and approach span piers, are summarized as follows:

- Design axial compressive load on micropiles = 237-kips/pile (max.)
- Design lateral load on single, vertical micropiles = 10-kips/pile (max.)

7.2.3 Design Parameters

See below for a table showing soil and rock properties that were incorporated into the proposed bridge abutment and approach span pier foundation design analyses.

Stratum	Unit Weight	Friction Angle	S_u[32]	$\varepsilon_{50\%}$[33]	k_{hi}[34]	RQD	UCS
	(pcf)	(degrees)	(psf)		(lbs/in^3)		(ksi)
1	115 to 125	30	-	-	90/60[35]	-	-
2	120 to 130	26 to 30	500	-	90/60	-	-
3	110 to 130	0	2000	0.005	-	-	-
Weathered Rock	145 to 150	38	-	-	225	0%	1.0
Competent Rock	155 to 160	-	-	-	-	50%	5.9 to 9.1

[31] Golder never received applied foundation loads for the proposed east / west abutments (i.e., Abutments 1 and 2).
[32] S_u = Undrained shear strength
[33] $\varepsilon_{50\%}$ = Strain at 50% of peak strength
[34] k_{hi} = Initial lateral modulus of subgrade reaction
[35] Value on left assumes unsaturated conditions, while the value on right assumes saturated (i.e., submerged) conditions.

7.2.4 Axial Capacity Analyses

Design methods presented in Section 10.8.3.5.4 of AASHTO (2012) were used to calculate the nominal (i.e., ultimate) axial capacity of single, vertical, rock-socketed micropiles, and it was assumed that only the rock socket side resistance contributed to micropile axial capacity (i.e., the end-bearing resistance at the micropile tip was neglected). Furthermore, a design factor-of-safety (FS) of 2.5 was applied to the computed micropile ultimate axial capacities to determine the allowable capacities of the micropiles.

At the bridge abutment (i.e., Abutments 1 and 2) and approach span pier (i.e., Piers 1 thru 4) locations, depths to bedrock were estimated utilizing the subsurface information within the vicinity of each abutment and pier foundation sub-structure. In particular, borings B-10, E-3, B-9, and B-8 represent subsurface conditions on the east side of the gorge (i.e., with the vicinity of Abutment 1, Pier 1, and Pier 2), while borings B-7, W-3, B-6, and B-5 represent subsurface conditions on the west side of the gorge (i.e., within the vicinity of Pier 3, Pier 4, and Abutment 2).

Axial capacity calculations were performed for varying pile diameters and rock socket depths utilizing a project-specific Excel spreadsheet. The accuracy of this spreadsheet was also verified utilizing hand calculations for a single pile diameter and rock socket length. Furthermore, these analyses assumed the micropile permanent steel casings would be drilled from bottom-of-pile-cap (BPC) to top-of-competent-rock (TCR), and rock sockets would be drilled further into competent bedrock (i.e., beneath the permanent casing).

With design input from M&M, the preferred, selected micropile design details were as follows:

- **Permanent Casings:** a) embed into reinforced-concrete pile caps; b) extend from BPC to TCR; and c) have nominal outside diameters of 12.75-inches (min.) and sidewall thicknesses of 0.5-inch (min.).

- **Rock Sockets:** a) drill 15-feet (min.) into competent bedrock (i.e., beneath bottoms of permanent casings); and b) have nominal diameters of 11.75-inches (min.).

- **Central reinforcing bars:** Insert centralized, single, vertical, Grade 75 (min.), #18 (i.e., nominal area of at least 4-square-inches) steel reinforcing bars into each micropile.

- **Cement Grout:** Infill the annual space between rock socket, permanent casing, and central reinforcing bar with at least 4,500-psi (28-day) cement grout.

In addition, magnitudes of micropile settlements were estimated in general accordance with AASHTO (2002), and the results of these computations indicate that single, vertical micropiles, used to support the proposed bridge abutments and approach span piers, would settle less than 0.5 inches.

7.2.5 Lateral Capacity Analyses

Lateral capacities and movements of single, vertical micropiles were analyzed using the LPILE (Ver. 2012.6.34) computer software, as developed and distributed by Ensoft. In addition, lateral movements of pile groups were further analyzed using the GROUP (Ver. 8.0.15) computer software, as developed and distributed by Ensoft.

LPILE analyses were conducted for each pier and abutment location assuming both fixed- and free-head[36] conditions. In general, lateral pile capacities will be dependent on the structural pile cap design, and fixed-head design conditions will be higher than that corresponding to free-head design conditions. However, actual lateral pile capacities will fall somewhere between the computed fixed- and free-head design conditions.

Allowable lateral micropile capacities (i.e., loads) were obtained by applying a design factor-of-safety of 2.5 to the computed ultimate lateral loads for given lateral head displacements (e.g., 0.5- and 1.0-inch). In addition, maximum allowable bending moments within the pile were obtained using LPILE to compute the bending moment induced within the micropile, under the applied allowable lateral load for given lateral head displacements.

Based on the preferred, selected 12.75-inch-diameter micropile design details (see Section 7.2.4 herein), the table below summarizes and presents the computed ultimate / allowable lateral load and bending moment capacities, at the micropile head, within single, vertical micropiles for two (2) assumed lateral micropile head displacement values (i.e., 0.5- and 1.0-inch).

Structure	Lateral Head Displacement	Free-Head Condition[37]				Fixed-Head Condition[36]			
		$P_{L,ult}$	$P_{L,all}$	M_{max} at $P_{L,ult}$	M_{max} at $P_{L,all}$	$P_{L,ult}$	$P_{L,all}$	M_{max} at $P_{L,ult}$	M_{max} at $P_{L,all}$
	(inch)	(kips)	(kips)	(kip-ft)	(kip-ft)	(kips)	(kips)	(kip-ft)	(kip-ft)
Abutment 1 (east)	0.5-inch	15.2	6.1	73.2	22.5	41.4	16.6	192.8	65.3
	1-inch	23.8	9.5	127.6	40.2	64.4	25.7	322.8	107.1
Pier 1	0.5-inch	30.5	12.2	105.4	33.9	69.1	27.6	263.2	83.6
	1-inch	38.4	15.4	172.3	42.8	81.4	32.5	348.6	97.9
Pier 2	0.5-inch	20.5	8.2	112.0	33.1	66.1	26.5	282.7	95.9
	1-inch	33.0	13.2	209.0	61.8	89.6	35.9	362.6	135.2
Pier 3	0.5-inch	20.5	8.2	128.2	41.8	69.0	27.6	290.7	102.8
	1-inch	34.5	13.8	232.4	80.2	90.4	36.2	348.5	138.1

[36] A fixed-head condition assumes zero rotation at pile head, while a free-head condition assumes zero moment at pile head.
[37] $P_{L,ult}$ and $P_{L,all}$ correspond to ultimate and allowable, respectively, lateral load values at the micropile head for the assumed lateral head displacements noted. M_{max} represents the maximum bending moment computed within the micropile at the noted (ultimate vs. allowable) lateral load values.

Structure	Lateral Head Displacement	Free-Head Condition[37]				Fixed-Head Condition[36]			
		$P_{L,ult}$	$P_{L,all}$	M_{max} at $P_{L,ult}$	M_{max} at $P_{L,all}$	$P_{L,ult}$	$P_{L,all}$	M_{max} at $P_{L,ult}$	M_{max} at $P_{L,all}$
	(inch)	(kips)	(kips)	(kip-ft)	(kip-ft)	(kips)	(kips)	(kip-ft)	(kip-ft)
Piers 4	0.5-inch	423	169	348.5	103.6	631	252	348.5	177.3
	1-inch	415	166	348.4	92.8	626	250	348.4	159.2
Abutment 2 (west)	0.5-inch	15.3	6.1	73.3	22.4	41.4	16.6	193.2	64.5
	1-inch	23.8	9.5	127.4	40.1	64.4	25.8	323.8	106.8

GROUP analyses were only performed for the proposed approach span piers (i.e., Piers 1 thru 4), given applied foundation loads were not provided for the proposed bridge abutments (i.e., Abutment 1 and 2), and said analyses were conducted assuming both fixed- and free-head design conditions, and pile cap embedment effects were also evaluated. In summary, the results of these GROUP analyses indicated the following:

- Free-head pile cap movement was less than 0.25-inch
- Fixed-head pile cap movement was less than 0.10-inch
- Embedded pile cap movements generally ranged between 1.5 and 3.0 times less than that of non-embedded pile caps

7.2.6 Downdrag

Per AASHTO (2012), downdrag forces on deep foundation can occur when, where:

1) Sites are underlain by compressible deposits, such as clays, silts, or organic soil materials
2) Fill materials will be or have been recently placed adjacent to deep foundations (i.e., piles or shafts), such as in the case for bridge approach embankment fills
3) The groundwater surface is substantially lowered
4) Liquefaction of loose sandy soil can occur

Furthermore, if soil mass settlements, relative to the pile/shaft movement, are equal to or greater than 0.4-inch, AASHTO (2012) indicates that downdrag forces can fully develop and should be accounted for in the requisite pile/shaft design analyses.

Based on completed site-specific liquefaction analyses (see Section 7.5 herein), the potential for liquefaction at the project site should be negligible. In addition, Golder cannot envision a reasonable design scenario where groundwater within the overburden would be "*substantially lowered*". Hence, Golder does not believe said liquefaction- or groundwater-induced downdrag forces are applicable to the proposed bridge abutment and approach span pier foundation designs.

Golder does believe that the above-noted downdrag conditions #1 and #2 are applicable to the subject project, and should be further evaluated, as part of the proposed bridge abutment and approach span pier foundation design analyses.

With respect to the proposed approach span piers (i.e., Piers #1 through #4), proposed grades around / above said piers will remain the same as the existing grades, or be lowered. Hence, Golder does not believe there is an appreciable potential for the development of downdrag on said pier foundations, and no further downdrag analyses were performed for these pier foundations.

However, there is an elevated potential for the development of downdrag forces on Abutments 1 and 2, due to the construction of the proposed approach railway embankments. In addition, Abutment 1 is underlain by a 12- to 15-foot-thick, stiff-to-hard clay layer (i.e., Stratum 3) that will compress under the applied railway embankment surcharge loads. Hence, the above-noted downdrag conditions #1 and #2 are applicable to the design of the proposed bridge abutment (Abutments 1 and 2) foundations.

In general, soil mass settlements comprise the following two (2) components: a) *"immediate settlements",* which predominantly are associated with the cohesionless materials atop Stratum 3 (i.e., embankment fill and Strata 1 and 2); and b) *"consolidation settlements",* which are primarily associated with Stratum 3. That said, the sequence of the proposed railway embankment construction will have an impact on whether, when, and to what degree associated downdrag forces at/on Abutments 1 and 2 are mobilized.

If the entire (i.e., full-height) railway approach embankments were first completed and the Abutment 1 and 2 micropile foundations subsequently installed, this construction approach / sequence should pre-induce the *"immediate settlements"* associated with embankment fill and Strata 1 and 2, and this would effectively mitigate the development of downdrag forces on Abutment 2 (West Abutment). However, this construction approach / sequence would not mitigate the *"consolidation settlements",* associated with Stratum 3, beneath the Abutment 1 (east abutment).

If immediate and consolidation settlements cannot be pre-induced before the Abutment 1 and 2 micropile foundations are installed, the magnitudes of downdrag on Abutments 1 and 2 were estimated to range between 80- and 85-kips/pile and 40- and 50-kips/pile, respectively.

That said, Golder believes the ground (i.e., soil and bedrock) has adequate resistive capacity (i.e., end-bearing, side resistance, or combined resistance) to support / carry the applied maximum axial design load (i.e., 237-kips/pile) plus upwards of 85-kips/pile of downdrag.

Given the proposed bridge abutments will be supported on a combination of battered / vertical micropiles, and said battered micropiles may be subject to downdrag, there will be additional bending moments induced in said battered micropiles resulting from vertical soil mass movements (i.e., settlement), which must be factored into the specified micropile structural capacity evaluations.

McGuire et al. (2010) provides a simplified design method, using the LPILE computer software, to estimate bending moments in battered piles subject to downdrag. Using this method, the additional downdrag-induced bending moments in the Abutment 1 (East Abutment) battered micropiles, assuming total soil mass settlements ranging between 1.0- and 1.5-inches, were estimated for both pinned- and fixed-head conditions as follows:

- **Pinned-Head Condition:**
  - Total Settlement = 1.0-inch: M_{max} = 43 kip-feet
  - Total Settlement = 1.5-inches: M_{max} = 61 kip-feet
- **Fixed-Head Condition:**
  - Total Settlement = 1.0-inch: M_{max} = 141 kip-feet
  - Total Settlement = 1.5-inches: M_{max} = 202 kip-feet

It should also be noted that these LIPLE analyses did not include any applied axial loads in/on the Abutment 1 battered micropiles. Hence, the above-noted bending moments are only associated with the soil above the battered pile and the vertical soil mass movement (i.e., settlement) resulting from downdrag on said battered micropiles.

7.3 Rock Slope Design

The proposed east / west rock cut slopes will be exposed to various rock slope stability and rockfall hazard risks due to, but not limited to, the following:

- Proposed rock cut slope geometries
- Steep to near vertical rock slopes with vertical heights on the order of 80 to 90 feet above the proposed arch skewback foundations
- Existing rock mass geologic conditions / mechanisms, which may eventually lead to future rockfall hazards
- Proximity of the new bridge's steel arch span superstructure and its arch skewback foundations situated adjacent to the proposed rock cut slopes

For the purpose of the subject project's rock slope design, rockfall hazard risks are generally categorized into the following three (3) types:

- ***Structural Risks:*** Risks associated with medium to large rock mass movements and/or rockfall, which would likely cause structural damage and/or loss-of-service to the new bridge's superstructure and foundations. Typical rock slope stabilization measures implemented to mitigate this type of risk include, but are not limited to, installation of rock anchors / dowels to secure / support unstable rock masses, excavation and removal of unstable rock masses, and installation of rock drains to reduce rock mass hydrostatic water pressures, which could contribute to destabilizing rock blocks.

- ***Non-Structural Risks:*** Risks associated with small to medium sized rockfall, which have the potential of impacting the new bridge's superstructure and foundations, but should not cause significant structural damage and/or loss-of-service. Typical rock slope stabilization measures implemented to mitigate this type of risk include, but are not limited to, additional rock anchors / dowels to support smaller-sized rock blocks, additional removal of unstable rock masses by scaling, application of shotcrete to mitigate raveling, and installation of rockfall drape nets to control the descent of rockfall.

- ***Inherent Risks:*** Risks associated with future rockfall due to continuing natural geologic processes (i.e., weathering and erosion). In general, this type of risk cannot be removed by engineered solutions, because some rock particle sizes tend to be too small and will pass through systems designed to catch said rock particles. This type of risk is typically mitigated by the implementation of routine inspection and maintenance programs.

7.3.1 Kinematic Analyses

A series of kinematic analyses, using the *Dips* software analysis package (Ver. 5.109; Rocscience Inc., 2012a), were performed for the proposed east / west rock cut slopes to: a) evaluate the proposed rock cut slope stability; and b) establish maximum allowable rock cut slope angles. In particular, Golder performed kinematic analyses for the following rock cut slope geometries / angles:

- 5V (vertical) to 1H (horizontal)
- 7.5V to 1H
- 10V to 1H

In general, kinematic analyses utilize geometric methods to assess different modes (e.g., planar, wedge, and toppling) of rock slope instability and/or failure. Furthermore, these analyses entail using stereographic projection techniques to plot three-dimensional orientation data, such as slope geometry, orientations of rock mass discontinuities, and discontinuity strength values (e.g., internal friction), as two-dimensional representations. These data then can be used to identify the number and nature of potentially adverse rock mass discontinuities, which may be exposed during excavation of the proposed rock cut slope.

See Appendix H for additional information regarding the above-noted kinematic analyses, and the results of said analyses.

7.3.2 Rock Dowels

To design the specified rock dowels, limit equilibrium techniques were employed to evaluate the sensitivity of possible failure modes / conditions to slope geometry and rock mass parameters (Hoek and Bray, 1981; Kliche, 1999; and Wyllie and Mah, 2004). Using this approach, magnitudes of additional tensional force required, assuming only untensioned rock dowels are installed, to maintain a factor-of-safety of 1.5 (i.e., dry, non-seismic conditions) were computed. Once these additional tensional forces were estimated, the number and location of the specified rock dowels was further designed.

Golder believes the use of untensioned, pattern rock dowels is appropriate for the proposed rock cut slopes, given the location of potential planar and wedge failures cannot be reasonably predicted and there is a need to reinforce the overall rock slope.

In addition, a single row / line of regularly spaced, steeply raked, untensioned rock dowels was added to the rock slope reinforcement design to: a) provide additional stability at the brow of the rock cut slopes; b) resist disturbance induced by construction activities; and c) resist long-term displacement from erosional or freeze-thaw forces.

7.3.3 Rock Drains

Rock drains will be used to relieve / reduce hydrostatic pressures within the rock mass, which could potentially develop into driving forces in/on the rock mass. Drains intercepting joints behind the rock face should lower water levels within the rock mass and behind the rock slope face, reducing weathering, rockfall, and ice formation.

7.3.4 Shotcrete

Shale / sandy-shale / shaly-sandstone layers were observed within the existing rock slope faces and encountered within the Maxim (1999) and Golder (2011) boreholes. Furthermore, these materials tend to be more susceptible to differential weather, which could lead to the generation of future rockfall, associated with small-scale toppling failure mechanisms.

While the proposed rockfall mitigation design includes the installation of a drape netting system, shotcrete may also be used to reduce the degree and extent of long-term differential weathering of the proposed rock cut slope surfaces. That said, it is envisioned that said shotcrete would only be used to cover those unsuitable materials susceptible to differential weathering (e.g., shale / sandy-shale / shaly-sandstone materials) to further resist against rock mass raveling.

However, the need for and location of shotcrete, if any, will be determined in the field, as the proposed rock cut slope excavation advance and the final rock cut slope surfaces are exposed.

7.3.5 Rockfall Drape Netting

Rockfall drape nets are commonly used to limit rockfall energies by controlling decent rates (i.e., velocities) of falling rockfall debris, and controlling / directing the vertical decent of rockfall into designated catchment areas. In general, rockfall drape nets are constructed as free-hanging meshes (i.e., drapes) supported along its upper rock slope crest, and lengths of coverage depend on slope geometries and the location and size of designated catchment ditches, if any.

For the subject project, the specified rockfall drape netting consists primarily of flexible, interconnected ring nets suspended by wire rope cable anchors, and the system includes a secondary double-twist wire mesh affixed atop said ring nets. The sides and bottom of the mesh will not be pinned to the rock face in order to allow accumulated rock fall debris to be controlled in its movement downslope.

The subject project's rockfall drape design incorporated a limit equilibrium approach, considering potential external loads on the drape in addition to the weight of the drape itself, followed by fabric type / size selection based on the slope geology, and then by anchor and system design details (WDOT, 2005). In general, the completed rock fall drape design method consisted of the following elements:

- Establish weight of typical drape panel, retained debris and snow and ice loads
- Determine anchor length and verify system component strength
- Conduct sensitivity analysis for slope conditions
- Check net strength versus cable strength and appropriate sag

Furthermore, the subject project's rockfall drape netting design assumed the following:

- Anchors consist of ¾-inch-diameter galvanized wire rope grouted into rock behind the crest of the slope, installed at inclinations of 45-degrees below horizontal, and spaced 17.5 feet apart
- Primary drape component consists of 3mm x 350mm galvanized ring nets
- Secondary drape consists of double twist 8x10 type wire mesh (placed over ring nets)
- Bedrock is sandstone with block sizes ranging in size from about 0.5 to 12 cubic-feet, with an average density of about 160-pcf
- Maximum of one (1) cubic-yard of rock material hanging in the drape
- Blocks larger than one (1) cubic-yard in size will be stabilized with rock dowels or scaled off the rock cut slope
- East and west slope are about 80-feet-high
- West slope has an angle of 84 degrees (i.e., 10V:1H)

- East slope has an angle of 79 degrees (i.e., 5V:1H)
- Ice buildup of about 0.5-foot-thick on the lower half of drape

Maintenance of the installed draped netting system should be minimal, consisting of inspections to evaluate the effectiveness of the drape and determine the amount of accumulated rock fall, either hung up in the drape or at the toe of slope. The integrity of the drape anchors should also be included during these inspections. In addition, if a large rockfall were to occur and is retained by the installed drape nets, the drape nets can be partially removed (i.e., taken apart), and subsequently reattached, to clear said rockfall from behind the drape nets.

7.4 Approach Embankments

Soil and rock slope stability analyses were performed using the two-dimensional, limit equilibrium software program *Slide* (Ver. 6.018; Rocscience, 2012c), and the generally accepted simplified Bishop method was utilized in these slope stability analyses. In general, the simplified Bishop method is regarded to be a non-rigorous method of analysis that employs the limit equilibrium "*method of slices*" to compute slope stability factors-of-safety.

For the subject project, both circular and non-circular slip surfaces were analyzed. *Slide* can search multiple, possible failure surfaces / planes, and present the surface / plane having the lowest computed factor-of-safety for the failure surfaces / planes analyzed.

Based on the *Slide* analysis results, the proposed railway approach embankment soil slopes have factors-of-safety equal to or greater than 1.3.

7.5 Seismic Liquefaction

As described in Section 6.1 herein, overburden conditions beneath the proposed railway alignment generally consist of topsoil and fill underlain by natural deposits of glacial gravel, sand, silt, clay, and weathered bedrock. Furthermore, Strata 1 and 2 include soils that could potentially be susceptible to seismic liquefaction. Hence, a site-specific liquefaction risk assessment was conducted.

In general, this site-specific liquefaction risk assessment of the overburden soils was conducted using the generally accepted, simplified, SPT N-value based method proposed by Seed and Idriss (1971), which was summarized, with modification (i.e., modified Seed Idriss procedure), by Youd et al. (2001). Furthermore, the site-specific liquefaction triggering analysis was performed for the 1,000-year return period ground motions (i.e., the hazard level of 7-percent probability of exceedance in 75 years) in accordance with AASHTO LRFD Bridge Design Specifications, 6th edition (AASHTO, 2012).

In addition, the site-specific acceleration response spectrum was determined, in accordance with the general procedure provided in AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd edition (AASHTO, 2011), using USGS/AASHTO Seismic Hazard Maps, as produced and distributed by the United States Geological Survey (USGS).

Golder completed site class determinations and liquefaction risk analyses using the SPT N-value data, from the B-series borings (i.e., B-01 through B-13), for the overburden materials (see Appendix B). Based on the results of the completed liquefaction analyses for the subject project, the potential for liquefaction should be negligible, under the 1,000-year return period hazard level, and calculated factors-of-safety against liquefaction, for selected representative borings using the modified Seed and Idriss procedure, were found to be greater than 1.9.

8.0 DESIGN RECOMMENDATIONS

This section presents Golder's project-specific foundation / geotechnical engineering design recommendations for the subject project. In particular, the following sections provide recommendations for the: a) rock slope excavation; b) rock slope reinforcement and rockfall mitigation; c) arch skewback foundations; d) bridge piers and abutments; e) railway approach embankments; f) construction monitoring; and g) long-term inspection and monitoring.

8.1 Rock Slope Excavation

Golder recommends that the proposed east and west rock cut slopes be excavated such that their back slopes are inclined as follows:

- **East Rock Cut Slope:** 5V (vertical) to 1H (horizontal)
- **West Rock Cut Slope:** 10V to 1H

See Figures 7, 8, and 12 for the approximate limits, geometry, and configuration for the proposed east and west rock cut slopes.

Rock excavations may be accomplished utilizing a combination of mechanical, chemical, and/or blasting techniques. However, Golder believes controlled rock blasting methods represent a reasonable approach for the subject project, and should afford Norfolk Southern the benefits of reduced construction durations and costs, relative to alternate mechanical and chemical rock removal methods.

8.2 Rock Slope Reinforcement and Rockfall Mitigation

As the requisite rock cut slope excavations advance vertically downward to the arch skewback foundation level, Golder recommends that the exposed rock cut slopes be further stabilized using a combination of rock drains, rock dowels, rock scaling, and shotcrete. The following sections provide additional details and recommendations relative to each of these design elements.

8.2.1 Rock Slope Dowels

Golder recommends that a series of passive (i.e., non-tensioned) rock dowels be installed within the exposed rock cut slope faces to reinforce and strengthen the rock mass. Furthermore, these rock dowels shall conform to the following:

- Rock slope dowels shall be installed on a typical staggered (i.e., offset), 10-foot (max.) spacing pattern (i.e., 1 rock dowel per 100-square-feet of exposed, excavated rock cut slope surface).
- Rock slope dowels shall: a) be installed in 3.5-inch-diameter (min.) drill holes; b) be drilled at 15-degree (typ.) declinations (i.e., below horizontal); and c) be fully grouted

using SikaGrout® 300 PT grout, which has a 28-day strength of 8,000-psi (min.), or approved equal.

- Install one (1) 40-foot-long (min.), 1.375-inch-diameter (#11 bar), Grade 150 (min.), "*hot dipped*" galvanized steel thread bar within each rock slope dowel drill hole.
- Provide PVC centralizers spaced on 7-foot-centers along the axis of each rock slope dowel and within 3-feet from the top and bottom of each drill hole.
- If grout loss is problematic, fabric "*socks*" may be used.
- Provide 5-inch-square (min.), 1.25-inch-thick (min.) steel bearing plates, spherical hex nut, and beveled washers to match the installed dowel inclinations.

See Figures 7, 8, 10, and 12 for approximate locations, spacing, and orientations of the specified rock slope dowels. Actual locations and quantities of rock slope dowels may deviate from that shown on Figures 7, 8, and 12, and additional rock slope dowels may be required, based on field observations made during and following completion of the requisite rock cut slope excavation activities.

8.2.2 Upper / Lower Brow Rock Dowels

Golder recommends that a series of passive (i.e., non-tensioned), steeply inclined rock dowels be installed along the east and west slope upper excavated rock cut brows and the west slope lower excavated rock cut brow to reinforce and strengthen these portions of the rock mass. Furthermore, these upper / lower brow rock dowels shall conform to the following:

- A total of eight (8) lower brow rock dowels shall be installed within the rock mass in front of the west arch skewback foundation, and said lower brow rock dowels shall be inclined toward the west arch skewback foundation at 75-degrees (typ.) below horizontal.
- Install a single-row of rock dowels along the east and west slope upper brows, and said rock dowels shall be spaced on 5-foot-centers along the entire upper brow of the excavated east and west rock cut slopes, and inclined away from the rock cut slope face at 75-degrees (typ.) below horizontal.
- Upper and lower brow rock dowels shall: a) be installed in 3.5-inch-diameter (min.) drill holes; and b) be fully grouted using SikaGrout® 300 PT grout, which has a 28-day strength of 8,000-psi (min.), or approved equal.
- Install one (1) 20-foot-long (min.), 1-inch-diameter (#8 bar), Grade 75 (min.), "*hot dipped*" galvanized steel thread bar within each upper / lower brow rock dowel drill hole.
- Provide PVC centralizers spaced on 7-foot-centers along the axis of each rock slope dowel and within 3-feet from the top and bottom of each drill hole.
- If grout loss is problematic, fabric "*socks*" may be used.
- Provide 5-inch-square (min.), 1.25-inch-thick (min.) steel bearing plates, spherical hex nut, and beveled washers to match the installed dowel inclinations.

See Figures 7, 8, and 9 for approximate locations, spacing, and orientations of the specified upper and lower brow rock dowels. Actual locations and quantities of upper and lower brow rock dowels may deviate from that shown on Figures 7 and 8, and additional rock slope dowels may be required, based on field observations made during and following completion of the requisite rock cut slope excavation activities.

8.2.3 Rock Drains

Golder recommends that a series of "*open*" rock drains be installed within the exposed rock cut slopes to reduce hydrostatic pressures at depth in the rock mass. Furthermore, these rock drains shall conform to the following:

- Within excavated back-slope surfaces, 50-foot-long (min.) rock drains shall be installed at the locations shown on Figure 12.
- Within excavated side-slope surfaces, 25-foot-long (min.) rock drains shall be installed at the locations shown on Figure 12.
- Rock drains shall be open, 3.5-inch-diameter (min.) holes drilled into rock at inclination angles of ten (10) degrees (typ.) above horizontal.

See Figures 7, 8, 10, and 12 for approximate locations, spacing, and orientations of the specified rock drains. Actual locations and quantities of rock drains may deviate from that shown on Figures 7, 8, and 12, and additional rock drains may be required, based on field observations made during and following completion of the requisite rock cut slope excavation activities.

8.2.4 Rock Scaling

Golder recommends that the exposed rock cut slope surfaces be hand-scaled, as necessary, to remove loose, unstable rock blocks / fragments, which could potentially move down-slope and represent future rockfall hazards. In general, the process of scaling involves the physical removal of loose, unstable rock blocks / fragments by the means of hand tools. To minimize damage to the exposed rock slope faces by over-scaling, mechanical scaling techniques should not be allowed.

8.2.5 Shotcrete

Golder recommends that steel fiber reinforced shotcrete be used, as appropriate and identified in the field, to resist erosion, differential weathering, and raveling of the scaled, final, exposed rock cut slope surfaces. If required, pneumatically applied shotcrete materials shall conform to the following:

- Prior to application of any shotcrete, rock surfaces to receive shotcrete shall be cleaned with water to ensure that the applied shotcrete will adequately adhere to the underlying rock.

- Apply 6-inch-thick (min.) wet or dry mix steel fiber reinforced shotcrete facings, with minimum 28-day compressive strengths of 4,000-psi (min.), to excavated, exposed rock cut slope surfaces that would be susceptible to differential weathering and/or erosion, as identified in the field.

- Install a series of short (i.e., 2- to 3-foot-long), 1-inch-diameter steel (i.e., #8 bar), Grade 75 (min.), "*hot dipped*" galvanized steel thread bar pins to support the applied shotcrete facings, as necessary. Furthermore, these steel pins shall be: a) installed on 10-foot-centers (max.); b) drilled 18- to 24-inches into rock; and c) embedded 4 inches (min.) into the applied shotcrete.

- Install 12-inch-wide (min.) geocomposite strip drains, spaced on 5- to 6-foot-centers (typ.) and centered between the installed rock dowels, to mitigate the buildup of hydrostatic pressures behind the applied shotcrete, as necessary.

- Install a series of weep holes/drains through the applied shotcrete facings to provide pathways for the drainage of water collected by the above-noted geocomposite strip drains.

However, it should be noted that the need for and actual locations of applied shotcrete, if any, will be determined and specified in the field, as the proposed rock cut slope excavation advances and the final rock cut slope surfaces are exposed.

8.2.6 Rockfall Drape Netting

Golder recommends that a continuous, double-layer (i.e., combined ring and wire mesh fabric nets fastened together) rockfall drape netting be installed on the excavated rock cut slope faces, which shall conform to the following:

- **Ring Nets:** Ring nets shall consist of 750-kiloJoule (kJ) nets made from interconnected steel rings, each ring with a nominal diameter of 13.8 inches (350 millimeters). Rings shall be composed of high tensile strength steel (i.e., minimum tensile strength of 203-ksi) wire, with a nominal diameter of 0.118-inch (3-millimeters), coiled into a loop with 7-loops-per-ring. Each ring shall connect to six (6) adjoining rings by passing through them. These ring nets shall be placed directly against the exposed rock cut slope surfaces, and fastened, using a series of shackles, to support cables and perimeter wire ropes.

- **Wire Mesh Fabric:** Wire mesh fabric shall be made from 11-gauge (3-mm-diameter), 8 by 10, double twist, galvanized steel wire mesh having a nominal mesh opening of 83-mm x 114-mm and conforming to ASTM A975, Style 1, as manufactured by Maccaferri, Inc. of Williamsport, Maryland or approved equal. This wire mesh fabric shall be placed directly atop the above-noted ring nets, and fastened, using a series of clips (i.e., fasteners), to the underlying ring nets.

- **Wire Rope Rock Anchor:** These anchors will be used to provide anchorage and support for the installed rockfall drape netting, and shall include: a) 10-foot-long (min.) ¾-inch-diameter (min.) double length galvanized wire rope; b) inclined 45-degrees (typ.) below horizontal; c) installed within 3.5-inch-diameter (min.) drill holes in rock; and d) be fully grouted using SikaGrout® 300 PT grout, which has a 28-day strength of 8,000-psi (min.), or approved equal.

- **Support Cables:** The top horizontal and vertical panel support cables for the drape nets shall be woven through the ring netting rings. Support cables shall consist of ¾-inch-diameter (min.) wire rope, galvanized, 6x19 class, IRWC (independent wire rope core) and EPIS (extra improved plow steel). Each support cable shall also have at least twenty-five (25) wires to a strand, and have a nominal breaking strength of 55-kips.

Installed rockfall drape netting shall be anchored at least five (5) feet laterally, horizontally behind the tops (i.e., upper brow or slope crest) of the east and west rock cut slopes, extend vertically down the east and west rock cut slopes, and terminate one (1) foot (typ.) above the bottoms of the exposed east and west rock cut slopes or about one (1) foot above the tops of the proposed arch skewback foundations.

See Figures 7, 8, 9, and 11 for the locations, lengths, and associated construction details in connection with the above-note rockfall drape netting.

8.3 Arch Skewback Foundations

Golder recommends the following in connection with the design and construction of the proposed arch skewback foundations:

- Both the east and west arch skewback foundations shall be founded directly on clean, sound, intact sandstone / siltstone, and sized allowing for a maximum allowable bearing capacity of 40-ksf.

- If any unsuitable shale / shaly-sandstone materials are encountered within the proposed arch skewback foundation bottom bearing surfaces, said shale / shaly-sandstone materials shall be removed, and the resulting void space infilled with mass (i.e., unreinforced) concrete having a minimum 28-day unconfined compressive strength of 3,000-psi.[38]

- The Contractor shall drill a minimum of four (4) confirmatory (NQ-sized) rock coreholes within the footprint area of both the east and west arch skewback foundations (i.e., a total of 8 confirmatory coreholes), which would be used to verify the rock quality beneath the proposed arch skewback foundations and delineate the limits and extents, if any, of any unsuitable shale / shaly-sandstone materials within the bottom bearing subgrade surfaces. These rock coreholes should be drilled from / between El. +1157 and El. +1168, and extend 40-feet (min.) vertically beneath the proposed arch skewback foundation bottom elevation (i.e., El. +1149). Upon completion of these confirmatory coreholes, the Contractor shall be infilled said coreholes with cement grout.

- The proposed arch skewback foundation concrete reinforcement cover distances (i.e., distance from reinforcement to the adjacent concrete-rock interfaces) should be increased by 1- to 2-inches to mitigate potential adverse effects associated with concrete placed against shale bedrock with pyrite concentrations elevated with respect to the underlying / overlying sandstone / siltstone rock.

- Unsuitable (e.g., shale and shaly-sandstone) materials should not be beneficially re-used to support any permanent structures on the subject project, but may be used within

[38] Golder further recommends that the Construction Documents incorporate an additional bid item for the potential over-excavation and infilling of any unsuitable, unacceptable shale and/or shaly-sandstone materials found within the bottoms of the proposed arch skewback foundations.

selected non-structural portions / sections of the proposed railway approach embankments, as approved by Golder.

See Figures 7 and 8 for additional information and details in connection with the proposed arch skewback foundations.

8.4 Abutment and Pier Foundations

Golder recommends the following in connection with the design and construction of the proposed bridge abutment and approach span pier foundations:

- Bridge abutments and approach span piers shall be founded on micropile (i.e., deep) foundations, which shall conform to the following:
  - **Permanent Casing:** Permanent casings shall be: a) installed utilizing "*duplex drilling*" methods; b) spaced at least three (3) pile diameters apart (i.e., no closer than 3 feet); c) embedded twelve (12) inches (min.) into pile caps; d) at least extend to TCR, as noted below; e) made of 50-ksi (min.) steel flush-joint or welded type pipe sections; f) have nominal outside diameters of 12.75-inch-diameter (min.) and sidewall thicknesses of 0.5-inch (min.).
  - **Rock Socket:** Drill 15-foot-long (min.), 11.75-inch-diameter (min.) rock sockets into competent bedrock (i.e., beneath TCR, as noted below).
  - **Central Reinforcing Bar:** Insert one (1) vertical steel reinforcing bar within the installed micropiles. Central reinforcing bars shall: a) have a nominal cross section areas of four (4) square-inches (min.); b) be made of Grade 75 (min.) steel; c) be in the form of continuous thread bars, as manufactured by Dywidag, SAS, Williams, or approved equal; and d) have PVC centralizes spaced ten (10) feet (max.) along entire length of central reinforcing bar.
  - **Cement Grout:** The annual space between rock socket, permanent casing, and central reinforcing bars shall be infilled with cement grout having a minimum 28-day compressive strength of 4,500-psi (min.).
- Micropiles shall be installed to the following design elevations:

Structure	Design Elevations (feet)			
	BPC	TOR	TCR	PT
Abutment 1 (East)	1297.0	1248.0	1242.0	1227.0
Pier 1	1262.0	1248.0	1242.0	1227.0
Pier 2	1256.0	1248.0	1242.0	1227.0
Pier 3	1253.5	1246.0	1242.0	1227.0
Piers 4	1255.0	1255.0	1247.0	1232.0
Abutment 2 (West)	1297.0	1270.0	1260.0	1245.0

Notes: 1) BPC = Bottom of pile cap
2) TOR = Top-of-rock
3) TCR = Top-of-competent-rock
4) PT = Pile tip

- Proposed approach span piers shall be supported on vertical micropiles, while the bridge abutments shall be supported on a combination of vertical and battered micropiles. Battered micropile angles shall not exceed 14-degrees (i.e., 12V : 3H), as measured from the battered micropile axis to vertical.

- Micropiles shall have maximum allowable axial compressive capacities of 237-kips/pile. Axial compressive load tests shall be performed on selected, representative vertical micropiles in accordance with ASTM D1143.

- Single, vertical micropiles shall have maximum allowable lateral capacities of 10-kips/pile. Lateral load tests, assuming fixed-head conditions, shall be performed on selected, representative vertical micropiles in accordance with ASTM D3966.

- Micropiles supporting the proposed east / west abutments (i.e., Abutments 1 and 2) shall be installed following completion of the proposed railway approach embankments to mitigate against the development of downdrag forces exerted on said micropiles.

In addition, Golder assumes M&M will evaluate and verify the structural capacity of the specified micropiles, subject to the applied design axial / lateral and downdrag[39] loads, supporting the proposed bridge abutments and approach span piers.

See Figures 13 for additional information and details in connection with the proposed micropile foundations.

8.5 Railway Approach Embankments

Golder recommends that the proposed sloped railway approach embankments, founded on native overburden soils, be constructed as follows:

- Proposed railway approach embankments shall be constructed utilizing "*unclassified*" and/or "*select*" fill materials, in accordance with NYSDOT Standard Specification Section 203.

- Fill materials used to construct the proposed railway approach embankments shall be placed, compacted, and tested in accordance with NYSDOT Standard Specification Section 203.

- If fine-grained materials are placed atop coarse-grained materials (e.g., sand/silt placed atop gravel/rockfill), reverse filters and/or separation barriers (e.g., woven or non-woven filter fabrics) shall be designed and installed, as appropriate, between dissimilar grain-sized materials to mitigate against the potential for internal erosion (i.e., vertical fines migration from overlying fine-grained materials into underlying coarse-grained materials).

- Prior to embankment construction, clearing, grubbing, and stripping activities shall be performed within the limits of the proposed railway approach embankments, and any identified "*unsuitable*" materials (e.g., historic fill, topsoil / organic, and deleterious materials) shall be removed.

[39] Assuming the specified micropiles will be installed following completion of the proposed railway approach embankments, the micropiles supporting the east bridge abutment (i.e., Abutment 1) will be subject to upwards of 85-kips/pile of downdrag.

- Upon completion of clearing, grubbing, and stripping activities, subgrade preparation activities shall be performed, which shall involve proof rolling of subgrade surfaces with a 15-ton (min.) vibratory compactor. If any weak subgrade soils are encountered, said "*unsuitable*" bearing materials shall be excavated and replaced.

- Railway approach embankments shall be constructed with typical 2 horizontal to 1 vertical (2H:1V) side slopes, and embankment side slopes shall not be steeper than 1.5H:1V. Furthermore, embankment slopes steeper than 2H:1V but flatter than 1.5H:1V shall include an 18-inch-thick layer of armor (e.g., stone aggregate or rip-rap) protection atop its slope surface.

8.6 Construction Monitoring

During construction of the subject project, Golder recommends that at least one (1) full-time, on-site, qualified geotechnical Professional Engineer (PE), licensed and registered in the State of New York, or an experienced field engineer / technician (non-PE), under the direction and responsible charge of a qualified geotechnical PE, licensed and registered in the State of New York, be retained by Norfolk Southern, as their designated geotechnical field representative.

Furthermore, Golder envisions that Norfolk Southern's designated geotechnical field representative would provide and perform the following, but not necessarily be limited to, services:

- Serve as a liaison between Norfolk Southern, the Engineer-of-Record, the Construction Manager, and the Contractor relative to the project's foundation and geotechnical-related construction activities.

- Document and maintain records (e.g., inspection reports, field notes, and photos / videos) of foundation and geotechnical-related construction activities.

- Observe compliance with Golder's project-specific geotechnical engineering design recommendations and the project's Construction Documents (i.e., drawings and specifications).

- Monitor the preparation of foundation, roadway, and approach embankment subgrades and bearing surfaces including, but not limited to, the removal of identified unsuitable bearing materials.

- Review foundation and geotechnical-related construction submittals.

- Review submitted controlled rock blasting plans, and monitor blasting activities.

- Confirm that fill materials meet the Contract Document requirements.

- Observe the placement and testing of approach embankment and roadway fill materials.

- Document and confirm that the character and nature of foundation subgrade bearing surfaces meet the Contract Document requirements.

- Monitor the installation of the specified micropiles, including the determination of when and where top-of-rock and top-of-competent-rock were achieved, to confirm compliance with the Contract Document requirements.

- Monitor micropile axial compressive and lateral load tests performed.

- Monitor and review field instrumentation (e.g., vibration monitoring and survey monitoring).

8.7 Inspection and Maintenance

Golder believes that the inspection and maintenance of the installed rockfall drape nets, rock dowels, and rock drains is integral to its rock slope design and the subject project's long-term performance. That said, Golder recommends that the following inspection and maintenance protocols be implemented, upon completion of the subject project:

- ***Inspection:*** The proposed east and west rock cut slopes should be inspected annually for the first five (5) years following construction and bi-annually (i.e., every other year) thereafter, or as deemed appropriate by Norfolk Southern. In addition, said inspections should, at a minimum, indicate any evidence of rock instability, erosion, and/or rockfall events, during the period since the prior inspection.

  Upon completion of each rock slope inspection event, detailed inspection reports should be prepared and provide, at a minimum, the following:

  - Findings and conclusions of field inspection surveys
  - Detailed photographic and/or video documentation of the slope conditions
  - Comparisons of conditions relative to previous inspection surveys
  - Identification of areas/zones of deterioration that may require maintenance
  - Preparation of drawings, photographs, videos, and/or sketches, as appropriate, showing the limits and details of required maintenance, as appropriate
  - Recommendations regarding when the next rock slope inspection should be scheduled

  Rock slope inspections should only be completed by qualified geotechnical engineers, geologic engineers, and/or engineering geologists that are properly trained to perform said inspections. In addition, these field inspections should include procedures requiring rope rappel or other specialty methods to closely inspect conditions and better assess the performance of the installed rock slope reinforcing elements.

- ***Maintenance:*** Maintenance of the subject rock slope could potentially involve, but not necessarily be limited to, any combination of the following:

  - Removal of vegetation
  - Additional rock scaling
  - Cleaning out and/or installing additional rock slope drains
  - Repairing and/or installing new rockfall drape nets and appurtances
  - Installation of supplemental and/or replacement rock dowels / wire rope anchors
  - Removal of accumulated soil / rock debris behind or in front of the installed rockfall drape nets

9.0 LIMITATIONS

This report was prepared, in accordance with generally accepted soil, rock, foundation, and geotechnical engineering practices, for: a) Norfolk Southern's and M&M's exclusive use in connection with the subject project; and b) specific applications on the subject project. Furthermore, the findings, conclusions, and recommendations contained herein were based on: 1) Golder's understanding of the project; and 2) relevant, associated project design information and details (e.g., foundation loads and design criteria) provided by M&M.

Recommendations included herein are contingent upon one another, and no recommendation should be taken nor followed independently of the others. Furthermore, the recommendations presented herein should not be altered, modified in part or in whole without Golder's written authorization. In addition, Golder's design recommendations should be incorporated into the project's construction drawings and technical specifications, as appropriate.

If changes to the nature, scope, and/or design of the subject project are proposed, planned, and/or implemented, the findings, conclusions, and recommendations presented herein should not be misconstrued to be valid, unless said design changes are reviewed and associated recommendations modified and/or verified, by Golder, in writing.

Subsurface borehole data only indicate conditions at specific locations and to depths penetrated, and these boreholes do not reflect soils strata or groundwater conditions and/or variations elsewhere. If variations in subsurface conditions are found to exist from those described, presented herein and/or noted, observed during construction, Golder should be notified, and the recommendations presented herein should be re-evaluated by Golder.

Stratification lines presented herein only represent approximate boundaries between differing geologic units, and actual transitions may be different from those depicted herein. Furthermore, interpreted stratigraphic boundaries were based on the available subsurface borehole data available to Golder, and the interpreted subsurface profiles shown on Figures 3 through 6 were created by laterally projecting the individual borehole data onto the proposed new railway centerline profile.

The professional engineering services rendered by Golder in connection with the subject project only included the foundation- and geotechnical-related aspects of the subsurface conditions found at the subject project site. Hence, the presence or implications of possible surface and/or subsurface contamination, resulting from previous activities or uses of the subject site and/or from the introduction of materials from off-site sources, are outside the terms of reference for this report, have not been investigated, and have not been addressed herein.

This report provides no warranties expressed or implied. In addition, Golder is not responsible for claims, damages, or liability arising from interpretations or reuse of subsurface information collected by, provided to, or made by others.

10.0 CLOSURE

Golder is pleased to have had the opportunity to prepare this ***Geotechnical Engineering Report*** for M&M in connection with the Norfolk Southern High Bridge (SR-361.66) Replacement Project, and looks forward to its continued participation on the project. If you have any questions, please feel free to contact the undersigned at (973) 645-1922.

Very truly yours,

GOLDER ASSOCIATES INC.

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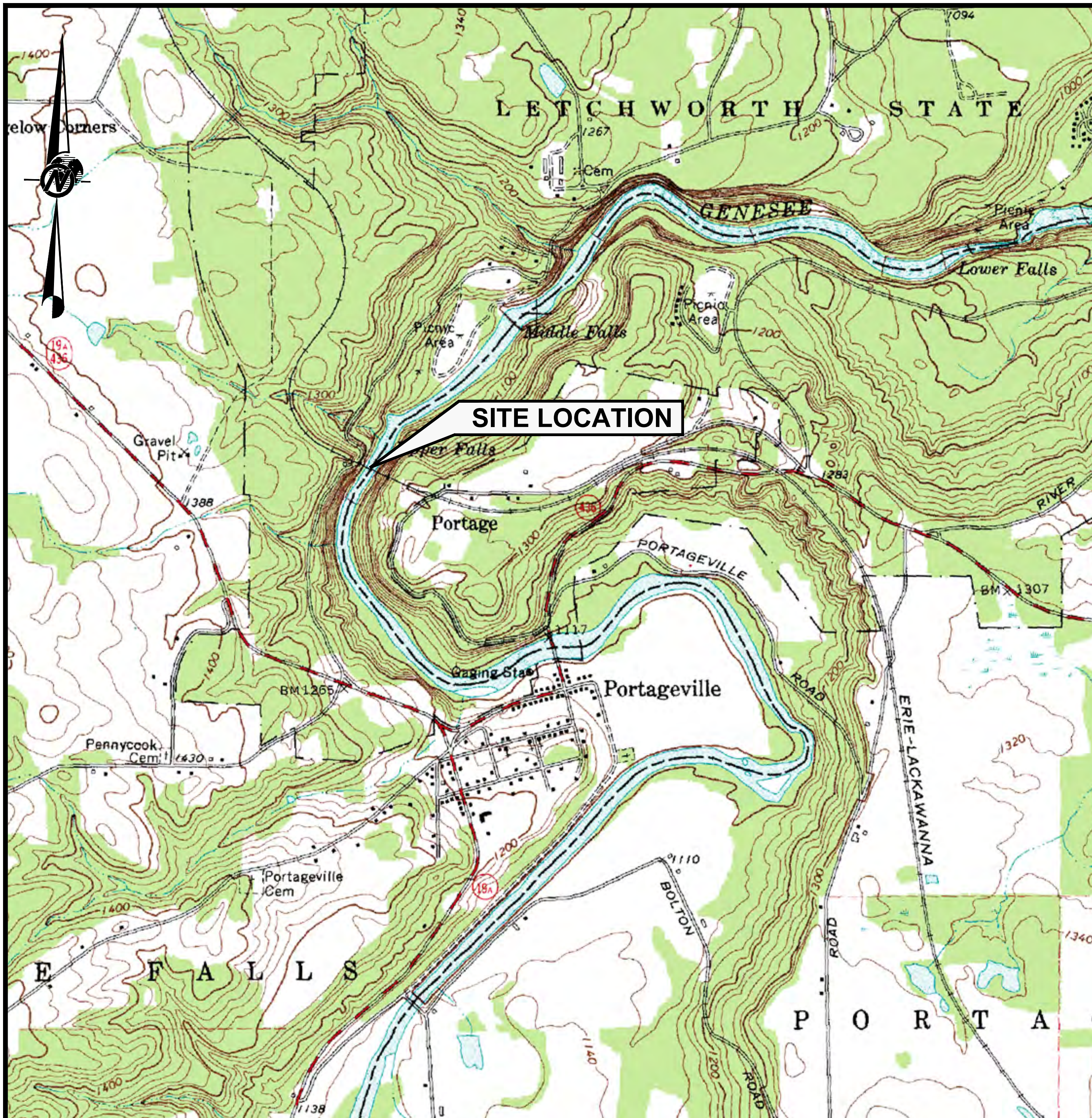
FIGURES



REFERENCE

1.) BASE MAP TAKEN FROM U.S.G.S. 7.5 MINUTE QUADRANGLE OF PORTAGEVILLE, NY DATED 1976, PHOTOINSPECTED 1976.

2000 0 2000

SCALE FEET

Golder Associates
Newark, New Jersey

NJ Authorization #24GA28029100

SCALE	AS SHOWN	TITLE	SITE LOCATION MAP
DATE	02/08/13		
DESIGN	DKL		
CADD	RG		
CHECK	DKL		
REVIEW	MFM	NORFOLK SOUTHERN CORPORATION	FIGURE 1

FILE No. 10386509D015

PROJECT No. 103–86509.01 REV. 0

Drawing file: 10386509D015.dwg Feb 08, 2013 – 9:15am

Borehole Information							
Boring No.	Track Station	Track Offset	Borehole Inclination (degrees)	Drilled Depth (feet)	Easting (feet)	Northing (feet)	Elevation (feet)
B-1	212+00	47L	Vertical	57.0	1290953.9	940471.8	1301.5
B-2	209+50	25L	Vertical	57.0	1291058.8	940233.7	1305.0
B-3	207+00	0L	Vertical	45.5	1291224.7	940041.4	1309.8
B-4	204+50	25R	Vertical	25.0	1291432.3	939907.8	1315.0
B-5	202+15	15R	Vertical	18.0	1291637.6	939799.0	1293.4
B-6	201+35	15L	Vertical	24.0	1291699.1	939739.7	1276.9
B-7	200+35	18L	Vertical	26.0	1291790.9	939699.9	1266.2
B-8	193+45	0L	Vertical	24.0	1292430.0	939439.5	1267.5
B-9	192+42	0L	Vertical	35.0	1292524.5	939398.6	1277.7
B-10	191+82	0L	Vertical	43.0	1292579.6	939374.7	1284.1
B-11	189+50	17.5R	Vertical	51.0	1292801.6	939307.3	1295.3
B-12	187+00	0L	Vertical	83.0	1293037.7	939216.9	1313.9
B-13	184+50	10L	Vertical	88.0	1293273.3	939133.5	1315.1
IB-1	199+80	10R	5	170.0	1291851.3	939701.0	1260.0
IB-2	199+90	10L	15	169.0	1291834.2	939686.6	1262.1
IB-3	193+90	10R	15	169.0	1292392.7	939466.5	1262.2
IB-4	194+00	5L	15	170.0	1292377.6	939456.8	1262.8
E-1	193+94	35.5R	Vertical	120.2	1292398.2	939491.9	1263.7
E-2	194+01	18R	Vertical	120.0	1292385.4	939478.6	1263.9
E-3	193+03	20R	Vertical	140.2	1292476.7	939440.7	1271.4
W-1	199+54	37.5R	Vertical	120.0	1291885.5	939716.2	1259.5
W-2	199+62	18.5R	Vertical	120.2	1291871.8	939701.4	1259.7
W-3	200+67	15L	Vertical	140.5	1291760.9	939712.8	1270.7

LEGEND

W-2 — MAXIM (1999) BORINGS

B-1 — GOLDER (2011) VERTICAL BORINGS

DIRECTION AND MAGNITUDE OF INCLINATION — IB-1 30° — GOLDER (2011) INCLINED BORINGS

100 — EXISTING RAILWAY ALIGNMENT

200 — PROPOSED RAILWAY ALIGNMENT

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, DEPTHS, AND OFFSETS ARE APPROXIMATE.

2.) SEE APPENDICES A AND B FOR COPIES OF MAXIM (1999) AND GOLDER (2011) BORING LOGS, RESPECTIVELY.

3.) SEE FIGURES 3 THROUGH 6 FOR INTERPRETED SUBSURFACE PROFILES A–A', B–B', C–C', AND D–D', RESPECTIVELY.

4.) AS–DRILLED BOREHOLE LOCATIONS WERE NOT SURVEYED FOR THE GOLDER (2011) BORINGS. ACTUAL BOREHOLE LOCATIONS WERE ESTABLISHED IN THE FIELD BY MEASURING DISTANCES OFF A SERIES OF FIXED SURVEY CONTROL POINTS, AS SURVEYED BY OTHERS, ALONG THE PROPOSED RAILWAY ALIGNMENT.

5.) GROUND SURFACE ELEVATIONS FOR THE GOLDER (2011) BORINGS WERE INTERPOLATED, BASED ON THE TOPOGRAPHIC CONTOURS SHOWN HEREIN.

6.) DRILLED DEPTHS REPORTED FOR BORINGS IB–1 THROUGH IB–4 REPRESENT LENGTHS ALONG THE INCLINED BOREHOLE AXIS.

7.) BOREHOLE INCLINATIONS ANGLES ARE MEASURED RELATIVE TO THE VERTICAL PROJECTION OF EACH ASSOCIATE BOREHOLE LOCATION.

REFERENCES

1.) MAXIM (1999) BOREHOLE INFORMATION TAKEN FROM REPORT, TITLED "PRELIMINARY GEOTECHNICAL EVALUATION – CONRAIL BRIDGE NO. 361.66", AS PREPARED BY MAXIM TECHNOLOGIES OF NEW YORK, INC., DATED MARCH 1999.

2.) BASE AMP CONTENTS COMPILES FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, AND 08049_FEA_ROW.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

3.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

4.) ELEVATIONS AND VERTICAL DATUM REFERENCES THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

100 0 100 200
SCALE FEET

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE

BORING LOCATION PLAN

NJ Authorization #24GA28029100

Golder Associates
Newark, New Jersey

PROJECT No.	103–86509.01	
DESIGN	DKL	02/08/13
CADD	RG	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

FILE No. 10386509D016
SCALE AS SHOWN REV. 0

FIGURE 2

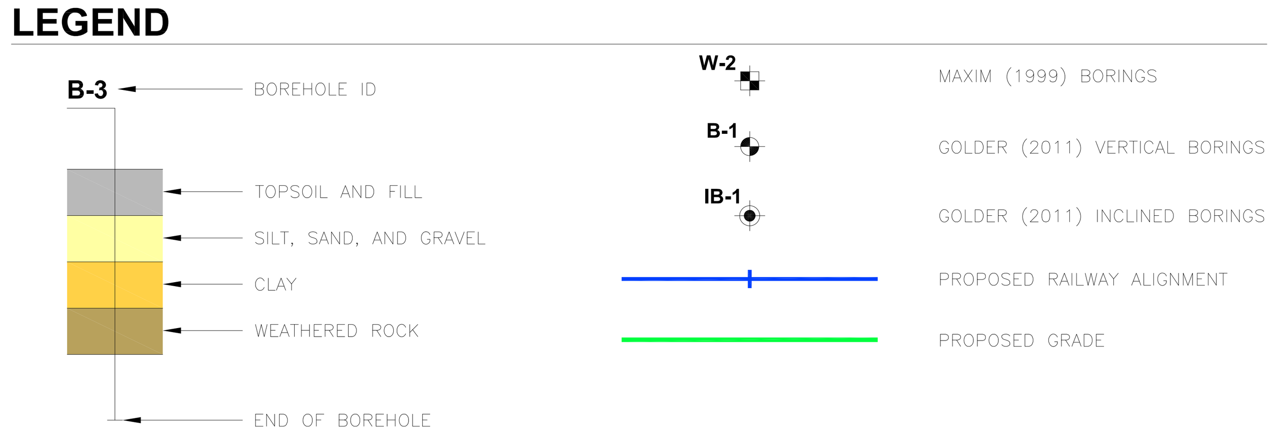
WEST - PLAN VIEW

40 0 40 80
SCALE FEET

WEST SECTION A-A'

40 0 40 80
SCALE FEET

LEGEND



NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) SEE FIGURE 2 FOR BORING LOCATION PLAN.

3.) SEE APPENDICES A AND B FOR COPIES OF MAXIM (1999) AND GOLDER (2011) BORING LOGS, RESPECTIVELY.

4.) THIS SUBSURFACE PROFILE WAS CONSTRUCTED ALONG THE PROPOSED RAILWAY ALIGNMENT CENTERLINE, AND EXISTING GROUND SURFACES SHOWN HEREIN WERE TAKEN FROM THE TOPOGRAPHIC CONTOURS, AS PRESENTED IN FIGURE 2.

5.) AS-DRILLED BOREHOLE LOCATIONS WERE NOT SURVEYED FOR THE GOLDER (2011) BORINGS. ACTUAL BOREHOLE LOCATIONS WERE ESTABLISHED IN THE FIELD BY MEASURING DISTANCES OFF A SERIES OF FIXED SURVEY CONTROL POINTS, AS SURVEYED BY OTHERS, ALONG THE PROPOSED RAILWAY ALIGNMENT.

6.) AVAILABLE SUBSURFACE INFORMATION ONLY INDICATES CONDITIONS AT SPECIFIC LOCATIONS AND TO DEPTHS PENETRATED, AND DO NOT REFLECT CONDITIONS ELSEWHERE.

7.) STRATIGRAPHIC LINES/BOUNDARIES SHOWN HEREIN HAVE BEEN INTERPRETED/INFERRED, BASED ON THE AVAILABLE SUBSURFACE INFORMATION, AND REPRESENT APPROXIMATE TRANSITIONS BETWEEN DIFFERING GEOLOGIC UNITS. ACTUAL TRANSITIONS BETWEEN GEOLOGIC UNITS MAY BE DIFFERENT FROM THOSE DEPICTED HEREIN.

8.) TOP OF ROCK REPRESENTS DEPTHS WHERE SPLIT-SPOON OR AUGER REFUSAL WAS ENCOUNTERED, AND DOES NOT REPRESENT TOP OF INTACT, SOUND (i.e., COMPETENT) BEDROCK.

9.) AS INDICATED ON FIGURE 2, NOT ALL OF THE AVAILABLE BORINGS WERE DRILLED ON THE PROPOSED RAILWAY CENTER LINE (i.e., BORING WERE OFFSET UPWARDS OF 47 FEET OFF CENTER LINE). THAT SAID, THE BOREHOLE INFORMATION SHOWN HEREIN WAS PROJECTED LATERALLY ONTO THE PROPOSED RAILWAY CENTER LINE.

REFERENCES

1.) MAXIM (1999) BOREHOLE INFORMATION TAKEN FROM REPORT, TITLED "PRELIMINARY GEOTECHNICAL EVALUATION – CONRAIL BRIDGE NO. 361.66", AS PREPARED BY MAXIM TECHNOLOGIES OF NEW YORK, INC., DATED MARCH 1999.

2.) BASE MAP CONTENTS COMPILED FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, 08049_FEA_ROW.DGN, PROPOSED CONTOURS 3D.DGN, AND PROPOSED.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

3.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

4.) ELEVATIONS AND VERTICAL DATUM REFERENCES THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT

NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE

INTERPRETED SUBSURFACE PROFILE A-A'

Golder Associates
Newark, New Jersey
NJ Authorization #24GA28029100

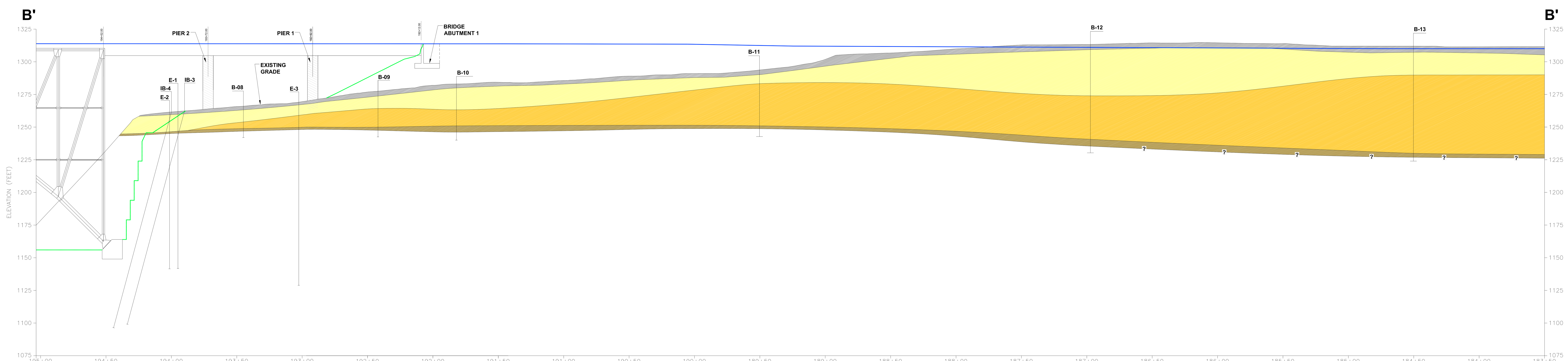
PROJECT No.	103-86509.01	
DESIGN	DKL	02/08/13
CADD	RG	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

FILE No. 10386509D017
SCALE AS SHOWN REV. 0

FIGURE 3

EAST - PLAN VIEW

40 0 40 80
SCALE FEET



EAST - SECTION B-B'

40 0 40 80
SCALE FEET

LEGEND

B-3	BOREHOLE ID
	TOPSOIL AND FILL
	SILT, SAND, AND GRAVEL
	CLAY
	WEATHERED ROCK
	END OF BOREHOLE
W-2	MAXIM (1999) BORINGS
B-1	GOLDER (2011) VERTICAL BORINGS
IB-1	GOLDER (2011) INCLINED BORINGS
	PROPOSED RAILWAY ALIGNMENT
	PROPOSED GRADE

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) SEE FIGURE 2 FOR BORING LOCATION PLAN.

3.) SEE APPENDICES A AND B FOR COPIES OF MAXIM (1999) AND GOLDER (2011) BORING LOGS, RESPECTIVELY.

4.) THIS SUBSURFACE PROFILE WAS CONSTRUCTED ALONG THE PROPOSED RAILWAY ALIGNMENT CENTERLINE, AND EXISTING GROUND SURFACES SHOWN HEREIN WERE TAKEN FROM THE TOPOGRAPHIC CONTOURS, AS PRESENTED IN FIGURE 2.

5.) AS-DRILLED BOREHOLE LOCATIONS WERE NOT SURVEYED FOR THE GOLDER (2011) BORINGS. ACTUAL BOREHOLE LOCATIONS WERE ESTABLISHED IN THE FIELD BY MEASURING DISTANCES OFF A SERIES OF FIXED SURVEY CONTROL POINTS, AS SURVEYED BY OTHERS, ALONG THE PROPOSED RAILWAY ALIGNMENT.

6.) AVAILABLE SUBSURFACE INFORMATION ONLY INDICATES CONDITIONS AT SPECIFIC LOCATIONS AND TO DEPTHS PENETRATED, AND DO NOT REFLECT CONDITIONS ELSEWHERE.

7.) STRATIGRAPHIC LINES/BOUNDARIES SHOWN HEREIN HAVE BEEN INTERPRETED/INFERRED, BASED ON THE AVAILABLE SUBSURFACE INFORMATION, AND REPRESENT APPROXIMATE TRANSITIONS BETWEEN DIFFERING GEOLOGIC UNITS. ACTUAL TRANSITIONS BETWEEN GEOLOGIC UNITS MAY BE DIFFERENT FROM THOSE DEPICTED HEREIN.

8.) TOP OF ROCK REPRESENTS DEPTHS WHERE SPLIT-SPOON OR AUGER REFUSAL WAS ENCOUNTERED, AND DOES NOT REPRESENT TOP OF INTACT, SOUND (i.e., COMPETENT) BEDROCK.

9.) AS INDICATED ON FIGURE 2, NOT ALL OF THE AVAILABLE BORINGS WERE DRILLED ON THE PROPOSED RAILWAY CENTER LINE (i.e., BORING WERE OFFSET UPWARDS OF 47 FEET OFF CENTER LINE). THAT SAID, THE BOREHOLE INFORMATION SHOWN HEREIN WAS PROJECTED LATERALLY ONTO THE PROPOSED RAILWAY CENTER LINE.

REFERENCES

1.) MAXIM (1999) BOREHOLE INFORMATION TAKEN FROM REPORT, TITLED "PRELIMINARY GEOTECHNICAL EVALUATION – CONRAIL BRIDGE NO. 361.66", AS PREPARED BY MAXIM TECHNOLOGIES OF NEW YORK, INC., DATED MARCH 1999.

2.) BASE MAP CONTENTS COMPILED FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, 08049_FEA_ROW.DGN, PROPOSED CONTOURS 3D.DGN, AND PROPOSED.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

3.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

4.) ELEVATIONS AND VERTICAL DATUM REFERENCES THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE
INTERPRETED SUBSURFACE PROFILE B-B'

Golder Associates
Newark, New Jersey
NJ Authorization #24GA28029100

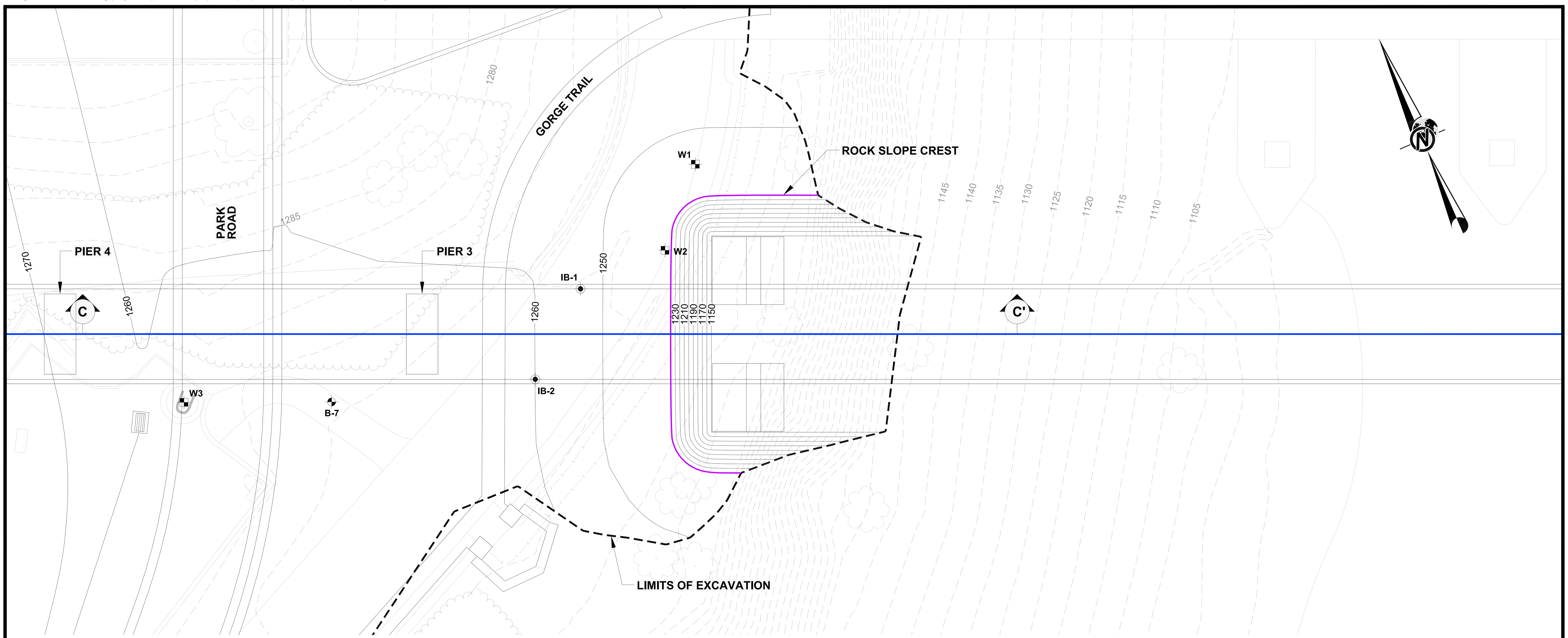
PROJECT No.	103-86509.01	
DESIGN	DKL	02/08/13
CADD	RG	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

FILE No. 10386509D018
SCALE AS SHOWN REV. 0

FIGURE 4

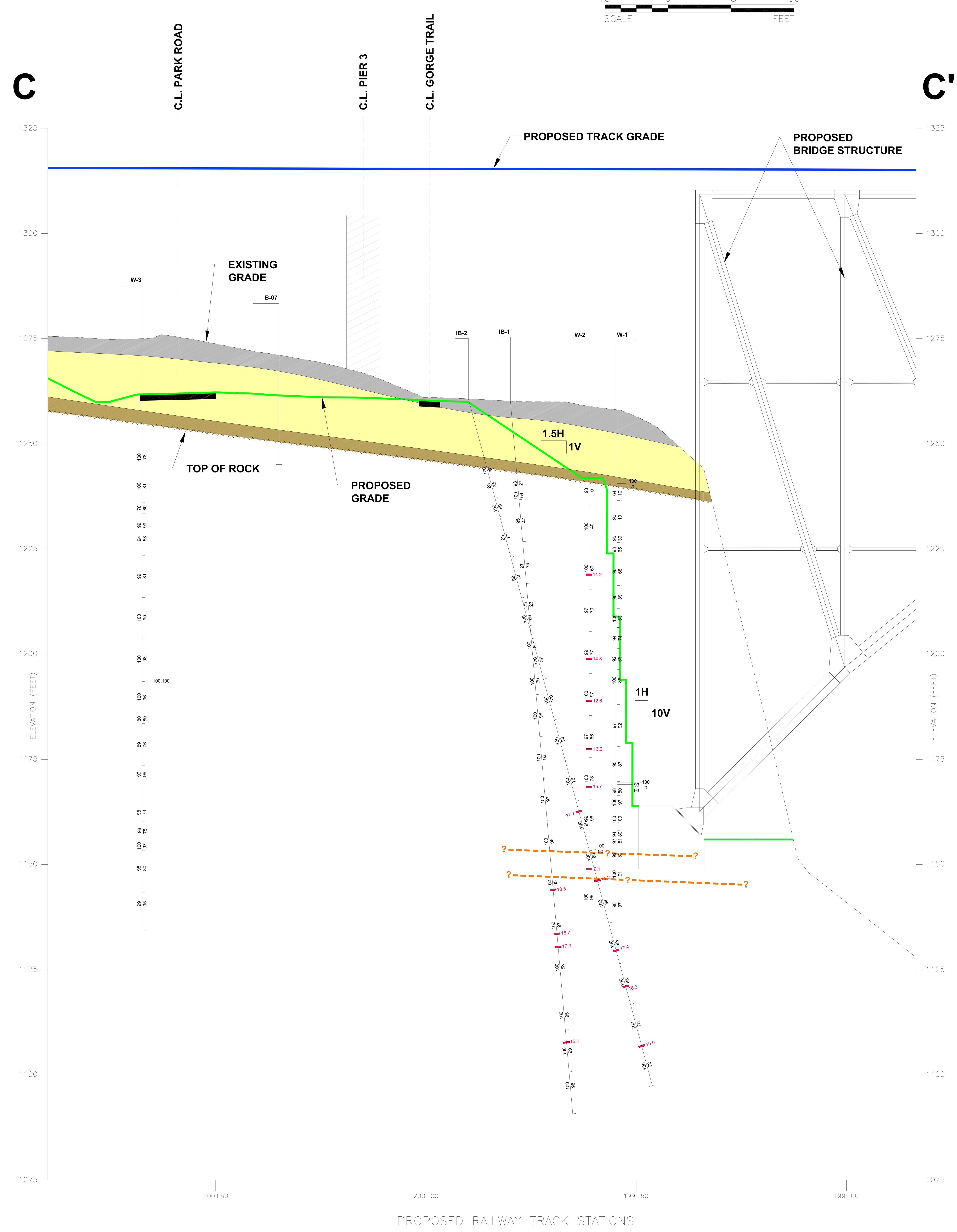
Drawing file: 10386509D018.dwg Feb 08, 2013 - 10:00am

Drawing File: 10386509D019.dwg | Layout: C | Modified: 01/23/13 1:35pm | Plotted: 02/08/13 10:04am | Plotted by: AMorales



WEST SLOPE - PLAN VIEW

15 0 15 30
SCALE FEET



WEST SLOPE - SECTION C-C'

15 0 15 30
SCALE FEET

LEGEND

- B-3 — BOREHOLE ID
- TOPSOIL AND FILL
- SILT, SAND, AND GRAVEL
- CLAY
- WEATHERED ROCK
- END OF BOREHOLE
- CORE RUN
- UNCONFINED COMPRESSIVE STRENGTH TEST ~ (ucs in ksi)
- GROUND WATER
- W-2 — MAXIM (1999) BORINGS
- B-1 — GOLDER (2011) VERTICAL BORINGS
- IB-1 — GOLDER (2011) INCLINED BORINGS
- PROPOSED RAILWAY ALIGNMENT
- 1110 — EXISTING CONTOURS
- 1150 — PROPOSED CONTOURS
- POTENTIAL SHALE / SHALY SANDSTONE LAYER

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) SEE APPENDICES A AND B FOR COPIES OF MAXIM (1999) AND GOLDER (2011) BORING LOGS, RESPECTIVELY.

3.) THIS SUBSURFACE PROFILE WAS CONSTRUCTED ALONG THE PROPOSED RAILWAY ALIGNMENT CENTERLINE, AND EXISTING GROUND SURFACES SHOWN HEREIN WERE TAKEN FROM THE TOPOGRAPHIC CONTOURS, AS PRESENTED IN FIGURE 2.

4.) AS-DRILLED BOREHOLE LOCATIONS WERE NOT SURVEYED FOR THE GOLDER (2011) BORINGS. ACTUAL BOREHOLE LOCATIONS WERE ESTABLISHED IN THE FIELD BY MEASURING DISTANCES OFF A SERIES OF FIXED SURVEY CONTROL POINTS, AS SURVEYED BY OTHERS, ALONG THE PROPOSED RAILWAY ALIGNMENT.

5.) AVAILABLE SUBSURFACE INFORMATION ONLY INDICATES CONDITIONS AT SPECIFIC LOCATIONS AND TO DEPTHS PENETRATED, AND DO NOT REFLECT CONDITIONS ELSEWHERE.

6.) STRATIGRAPHIC LINES/BOUNDARIES SHOWN HEREIN HAVE BEEN INTERPRETED/INFERRED, BASED ON THE AVAILABLE SUBSURFACE INFORMATION, AND REPRESENT APPROXIMATE TRANSITIONS BETWEEN DIFFERING GEOLOGIC UNITS. ACTUAL TRANSITIONS BETWEEN GEOLOGIC UNITS MAY BE DIFFERENT FROM THOSE DEPICTED HEREIN.

7.) TOP OF ROCK REPRESENTS DEPTHS WHERE SPLIT-SPOON OR AUGER REFUSAL WAS ENCOUNTERED, AND DOES NOT REPRESENT TOP OF INTACT, SOUND (i.e., COMPETENT) BEDROCK.

REFERENCES

1.) MAXIM (1999) BOREHOLE INFORMATION TAKEN FROM REPORT, TITLED "PRELIMINARY GEOTECHNICAL EVALUATION – CONRAIL BRIDGE NO. 361.66", AS PREPARED BY MAXIM TECHNOLOGIES OF NEW YORK, INC., DATED MARCH 1999.

2.) BASE MAP CONTENTS COMPILED FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, 08049_FEA_ROW.DGN, PROPOSED CONTOURS 3D.DGN, AND PROPOSED.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

3.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

4.) ELEVATIONS AND VERTICAL DATUM REFERENCES THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

FIGURE 5

REVIEW	MFM	02/08/13
CHECK	DKL	02/08/13
CADD	RG	02/08/13
DESIGN	DKL	02/08/13
REV. 0	SCALE AS SHOWN	
FILE No.	10386509D019	
PROJECT No.	103-86509.01	

TITLE
INTERPRETED SUBSURFACE PROFILE C-C'

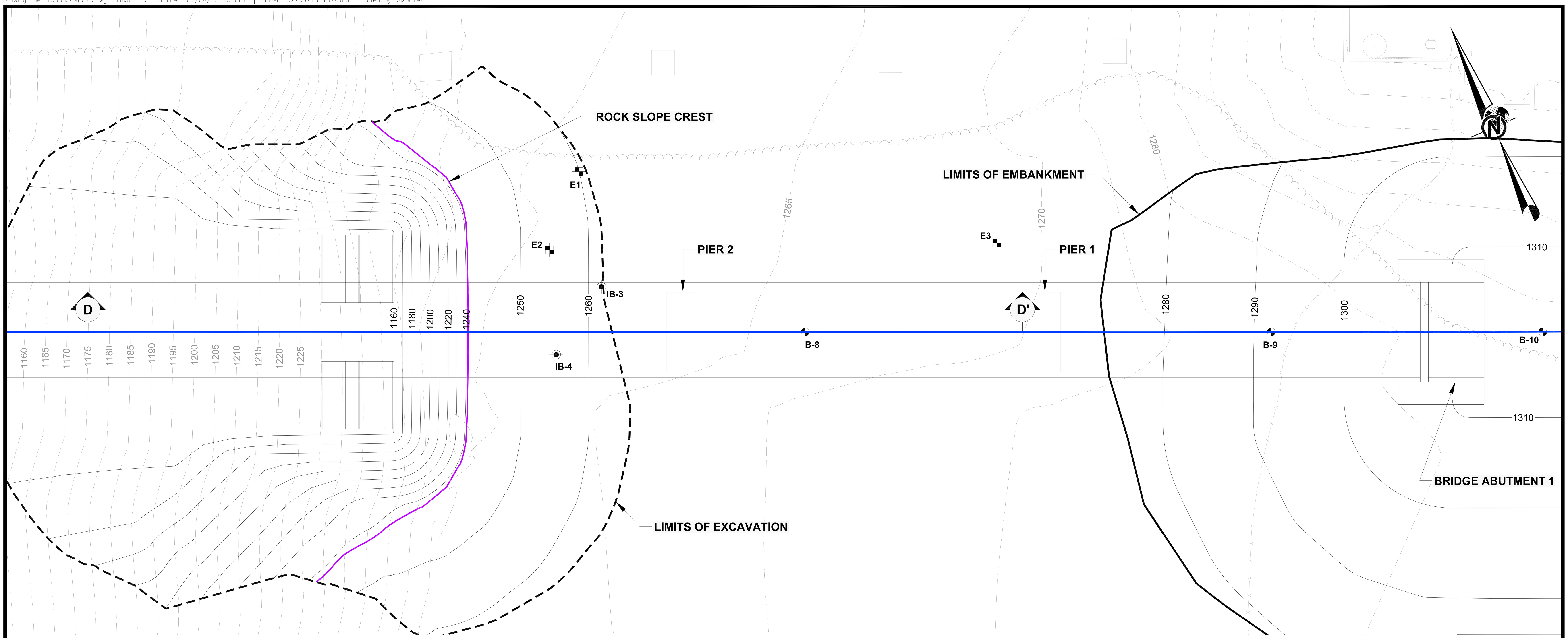
PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

NJ Authorization #24GA28029100

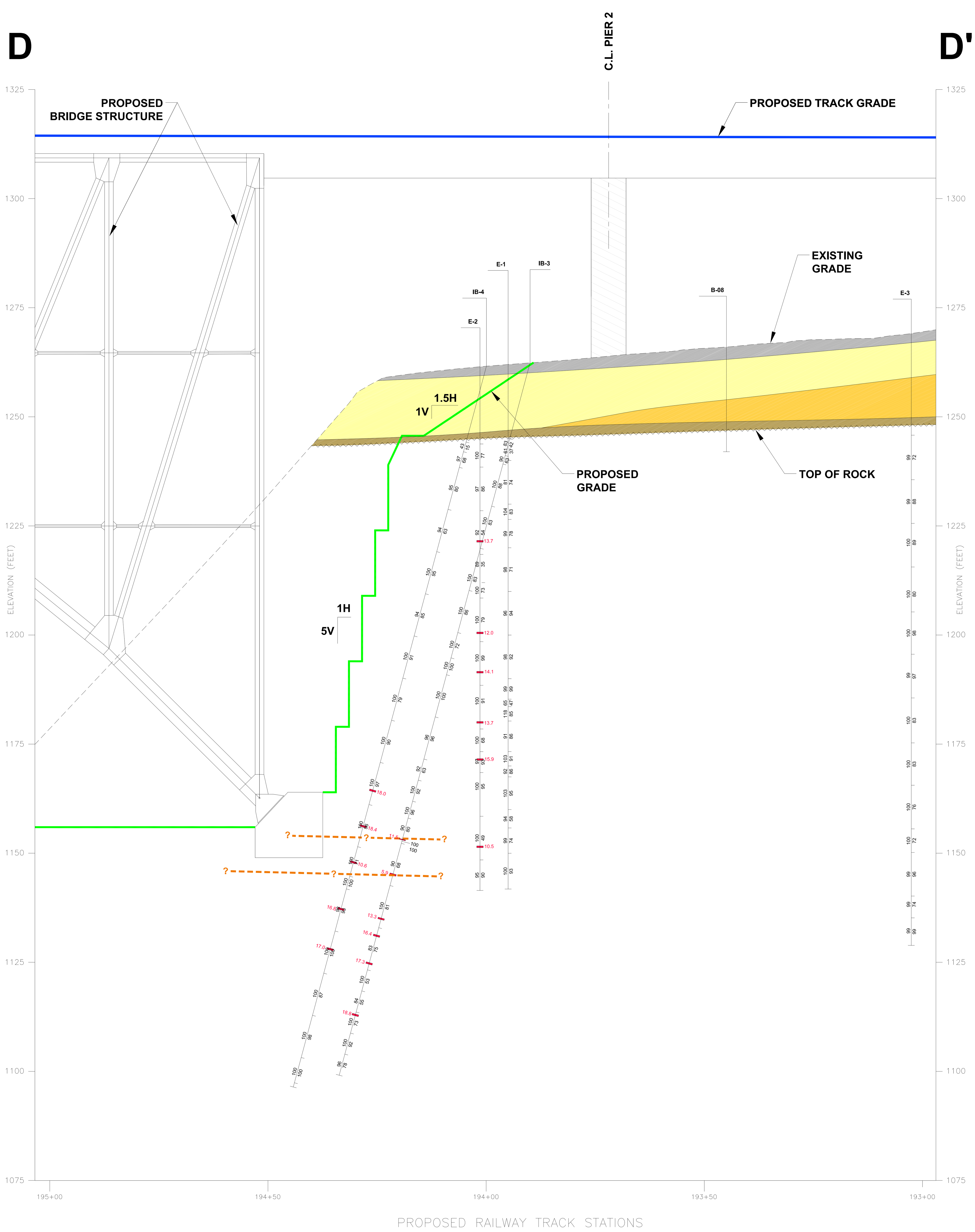
Golder Associates
Newark, New Jersey

Drawing File: 10386509D020.dwg | Layout: D | Modified: 02/08/13 10:06am | Plotted: 02/08/13 10:07am | Plotted by: AMorales



EAST SLOPE - PLAN VIEW

15 0 15 30
SCALE FEET



EAST SLOPE - SECTION D-D'

15 0 15 30
SCALE FEET

LEGEND

B-3 — BOREHOLE ID
TOPSOIL AND FILL
SILT, SAND, AND GRAVEL
CLAY
WEATHERED ROCK
END OF BOREHOLE
CORE RUN
UNCONFINED COMPRESSIVE STRENGTH TEST - (ucs in ksi)
GROUND WATER
W-2 — MAXIM (1999) BORINGS
B-1 — GOLDER (2011) VERTICAL BORINGS
IB-1 — GOLDER (2011) INCLINED BORINGS
PROPOSED RAILWAY ALIGNMENT
1110 — EXISTING CONTOURS
1150 — PROPOSED CONTOURS
POTENTIAL SHALE / SHALY SANDSTONE LAYER

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) SEE APPENDICES A AND B FOR COPIES OF MAXIM (1999) AND GOLDER (2011) BORING LOGS, RESPECTIVELY.

3.) THIS SUBSURFACE PROFILE WAS CONSTRUCTED ALONG THE PROPOSED RAILWAY ALIGNMENT CENTERLINE, AND EXISTING GROUND SURFACES SHOWN HEREIN WERE TAKEN FROM THE TOPOGRAPHIC CONTOURS, AS PRESENTED IN FIGURE 2.

4.) AS-DRILLED BOREHOLE LOCATIONS WERE NOT SURVEYED FOR THE GOLDER (2011) BORINGS. ACTUAL BOREHOLE LOCATIONS WERE ESTABLISHED IN THE FIELD BY MEASURING DISTANCES OFF A SERIES OF FIXED SURVEY CONTROL POINTS, AS SURVEYED BY OTHERS, ALONG THE PROPOSED RAILWAY ALIGNMENT.

5.) AVAILABLE SUBSURFACE INFORMATION ONLY INDICATES CONDITIONS AT SPECIFIC LOCATIONS AND TO DEPTHS PENETRATED, AND DO NOT REFLECT CONDITIONS ELSEWHERE.

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7.) TOP OF ROCK REPRESENTS DEPTHS WHERE SPLIT-SPOON OR AUGER REFUSAL WAS ENCOUNTERED, AND DOES NOT REPRESENT TOP OF INTACT, SOUND (i.e., COMPETENT) BEDROCK.

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3.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

4.) ELEVATIONS AND VERTICAL DATUM REFERENCES THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

FIGURE 6

REVIEW	MFM	02/08/13
CHECK	DKL	02/08/13
CADD	RG	02/08/13
DESIGN	DKL	02/08/13
REV. 0	SCALE AS SHOWN	
FILE No.	10386509D020	
PROJECT No.	103-86509.01	

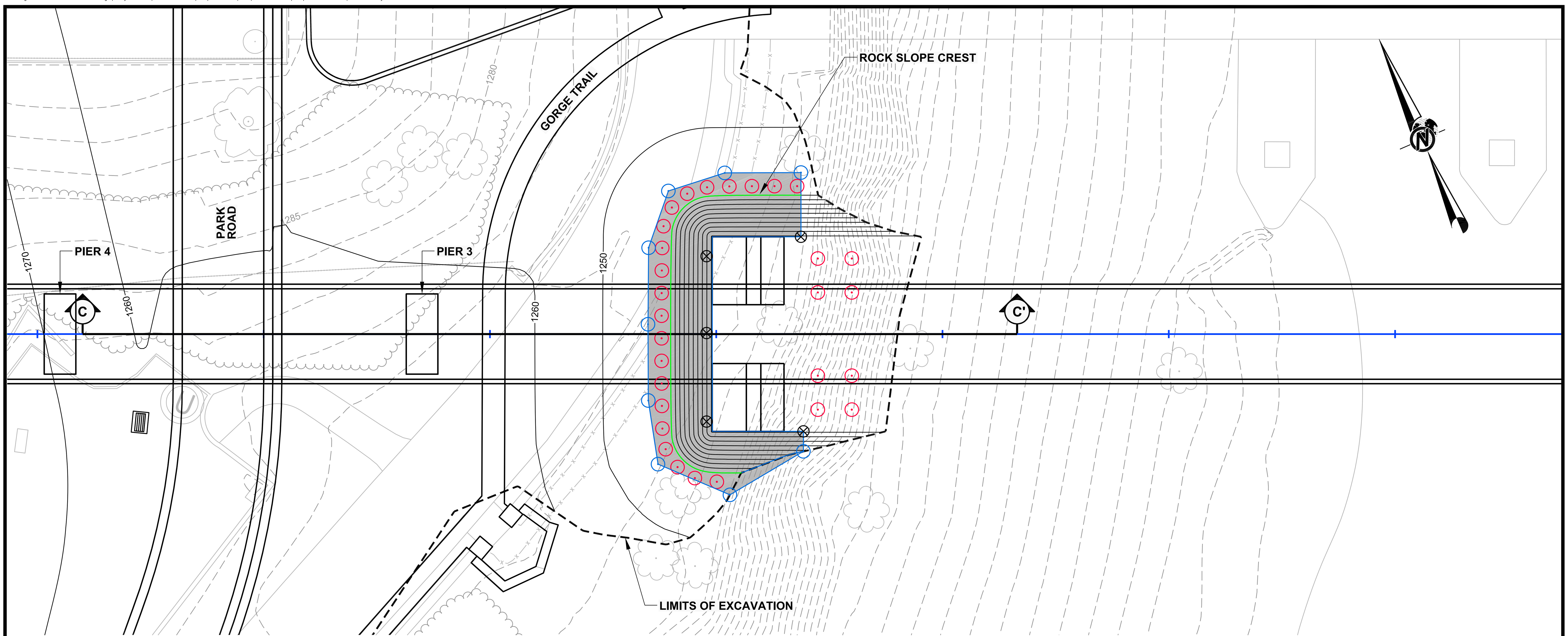
TITLE
INTERPRETED SUBSURFACE PROFILE D-D'

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

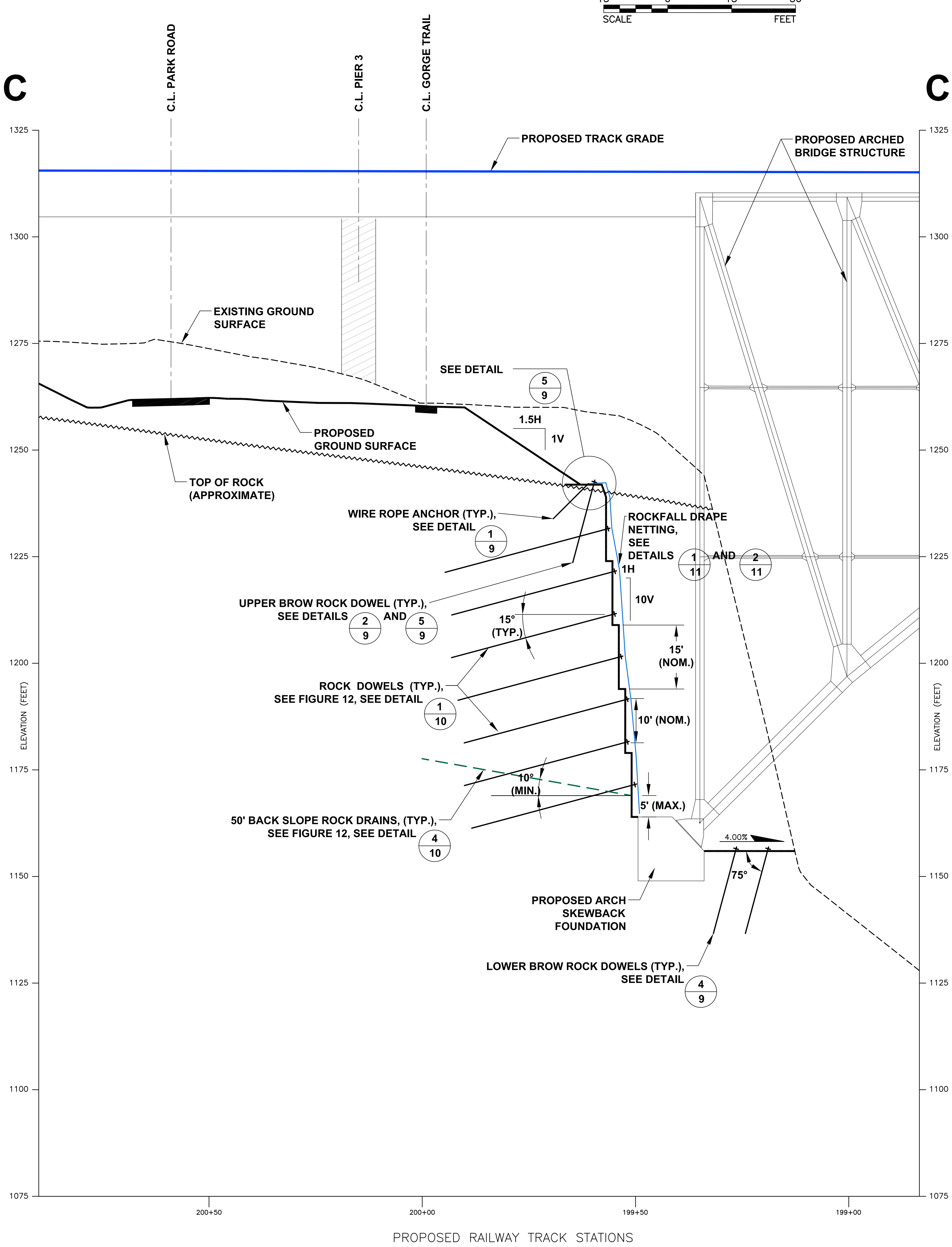
NJ Authorization #24GA28029100

Golder Associates
Newark, New Jersey



WEST SLOPE - PLAN VIEW

15 0 15 30
SCALE FEET



WEST SLOPE - SECTION C-C'

15 0 15 30
SCALE FEET

LEGEND

C	DETAIL OR CROSS SECTION DESIGNATION
2/9	DETAIL NUMBER / DETAIL SHEET
(blue circle)	WIRE ROPE ANCHOR, SEE DETAIL 1/9
(red circle)	UPPER/LOWER BROW ROCK DOWEL, SEE DETAILS 2/9 AND 4/9
(blue line)	PROPOSED RAILWAY ALIGNMENT
(dashed line)	EXISTING CONTOURS
(solid line)	PROPOSED CONTOURS
(shaded box)	ROCKFALL DRAPE NETTING, SEE DETALS 1/11 AND 2/11
(green dashed line)	ROCK DRAIN, SEE DETAIL 4/10
⊗	ROCK DRAIN, SEE DETAIL 4/10 AND SEE FIGURE 12

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) SEE FIGURE 12 FOR LOCATIONS OF 25' LONG SIDE SLOPE ROCK DRAINS AND 50' LONG BACK SLOPE ROCK DRAINS.

REFERENCES

1.) BASE MAP CONTENTS COMPILED FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, 08049_FEA_ROW.DGN, PROPOSED CONTOURS 3D.DGN, AND PROPOSED.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

2.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

3.) ELEVATIONS AND VERTICAL DATUM REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

FIGURE 7

PROJECT No.	103-86509.01	
FILE No.	103865090009	
REV. 0	SCALE	AS SHOWN
DESIGN	JRS	02/08/13
CADD	MPB	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

TITLE

WEST SLOPE STABILIZATION PLAN

PROJECT

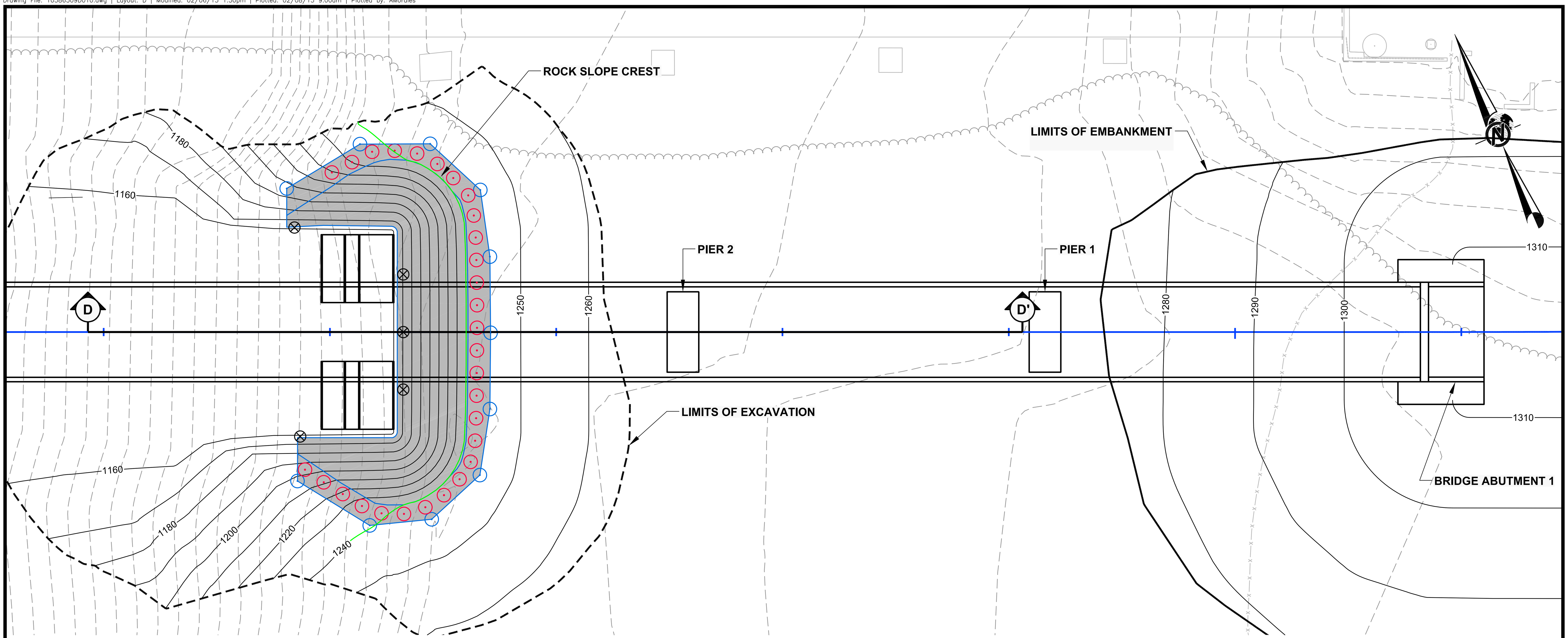
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

NJ Authorization #24GA28029100

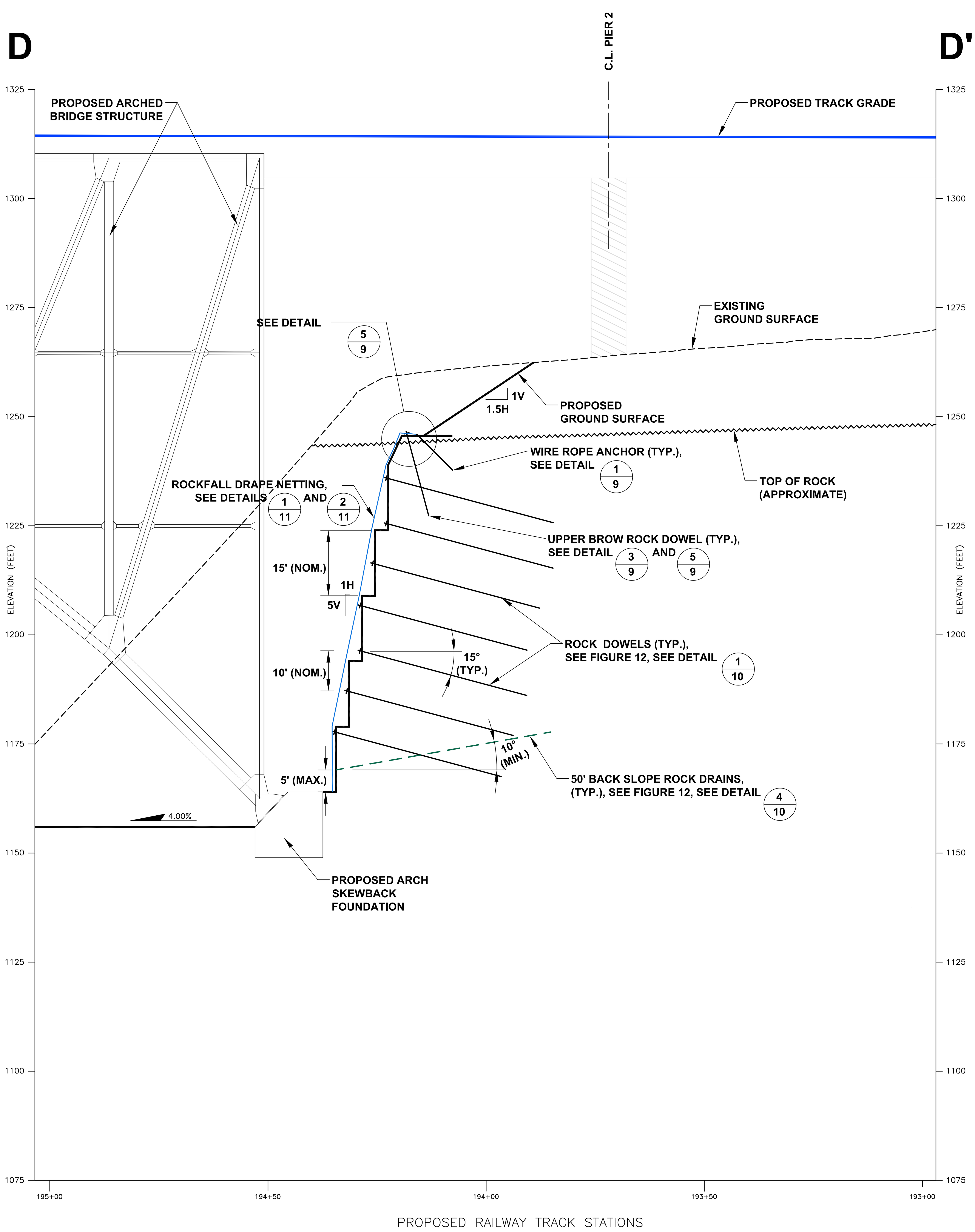
Golder Associates
Newark, New Jersey

Drawing File: 10386509D010.dwg | Layout: D | Modified: 02/06/13 1:50pm | Plotted: 02/08/13 9:00am | Plotted by: AMorales



EAST SLOPE - PLAN VIEW

15 0 15 30
SCALE FEET



EAST SLOPE - SECTION D-D'

15 0 15 30
SCALE FEET

LEGEND

- D — DETAIL OR CROSS SECTION DESIGNATION
- 2/9 — DETAIL NUMBER / DETAIL SHEET
- WIRE ROPE ANCHOR, SEE DETAIL 1/9
- UPPER BROW ROCK DOWEL, SEE DETAILS 3/9
- PROPOSED RAILWAY ALIGNMENT
- EXISTING CONTOURS
- PROPOSED CONTOURS
- ROCKFALL DRAPE NETTING, SEE DETAL 1/11 AND 2/11
- ROCK DRAIN, SEE DETAIL 4/10
- ROCK DRAIN, SEE DETAIL 4/10 AND SEE FIGURE 12

NOTES

1.) ALL LOCATIONS, ELEVATIONS, INCLINATIONS, AND DEPTHS ARE APPROXIMATE.

2.) FOR EAST ROCK CUT SLOPE, SECTIONS 1 AND 5 ON FIGURE 9 ARE THE MIRROR IMAGE TO THAT SHOWN ON FIGURE 9.

3.) SEE FIGURE 12 FOR LOCATIONS OF 25' LONG SIDE SLOPE ROCK DRAINS AND 50' LONG BACK SLOPE ROCK DRAINS.

REFERENCES

1.) BASE MAP CONTENTS COMPILED FROM MULTIPLE MICROSTATION FILES (I.E., BL_75 FT OFFSET.DGN, BL_EXIST.DGN, 08049_MAP_SUR_3DH.DGN, 08049_FEA_ROW.DGN, PROPOSED CONTOURS 3D.DGN, AND PROPOSED.DGN), AS PREPARED AND PROVIDED TO GOLDER BY MODJESKI AND MASTERS, INC.

2.) HORIZONTAL DATUM REFERENCES THE NEW YORK STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83).

3.) ELEVATIONS AND VERTICAL DATUM REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 0.522 FEET BELOW THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD29).

FIGURE 8

REVIEW	MFM	02/08/13
CHECK	DKL	02/08/13
CADD	MPB	02/08/13
DESIGN	JRS	02/08/13
REV. 0	SCALE AS SHOWN	
FILE No.	10386509D010	
PROJECT No.	103-86509.01	

TITLE

EAST SLOPE STABILIZATION PLAN

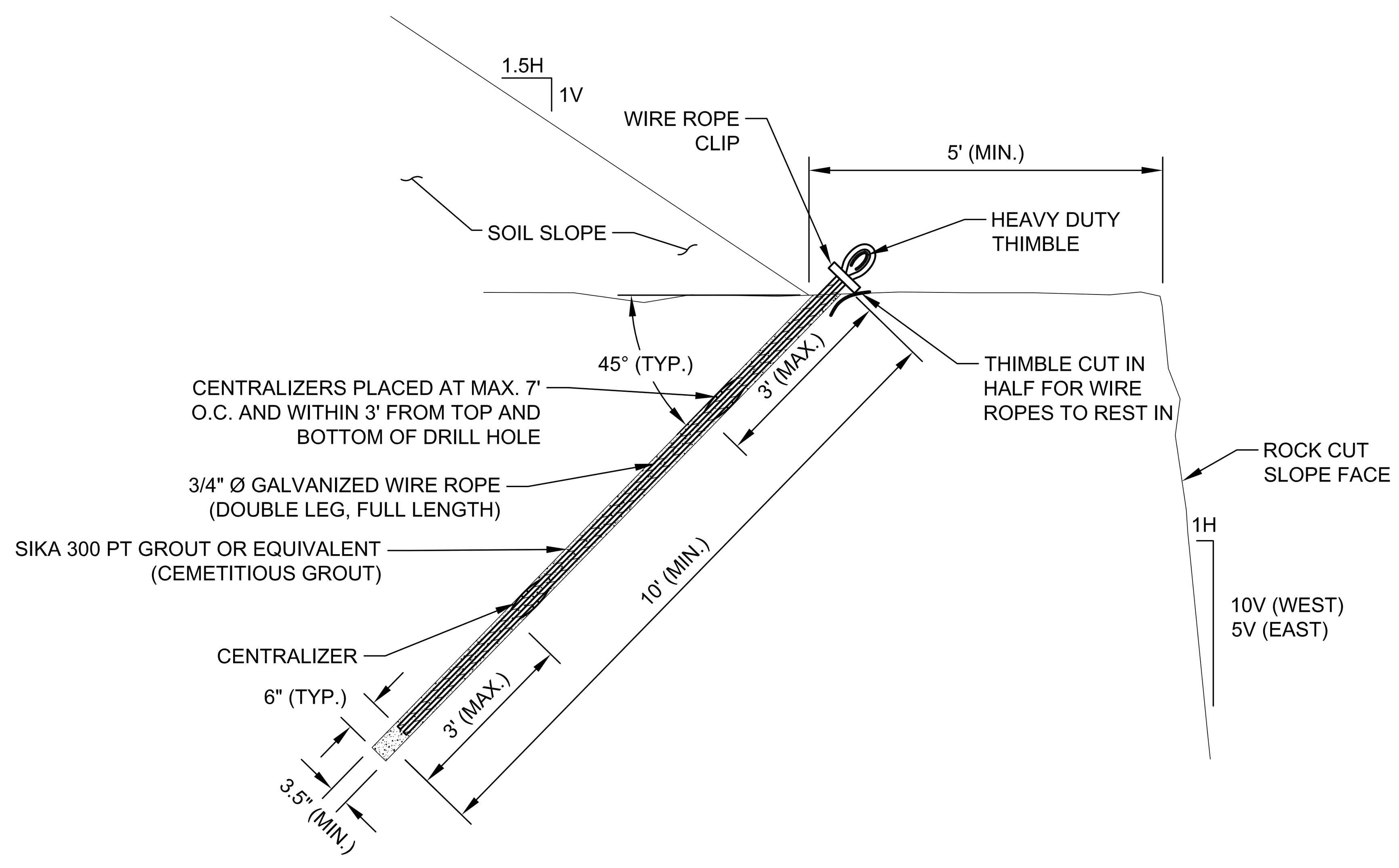
PROJECT

NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

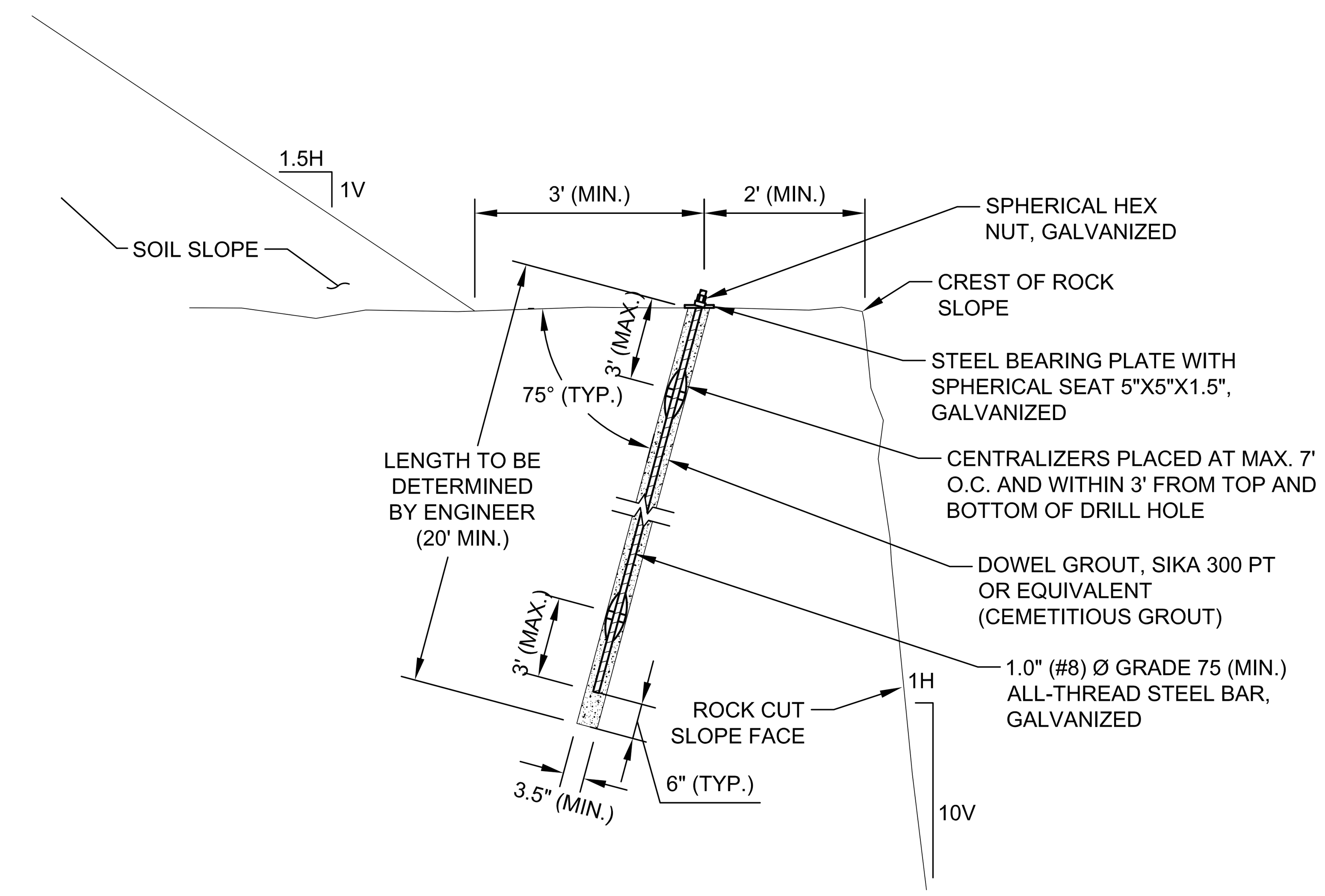
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

NJ Authorization #24GA28029100

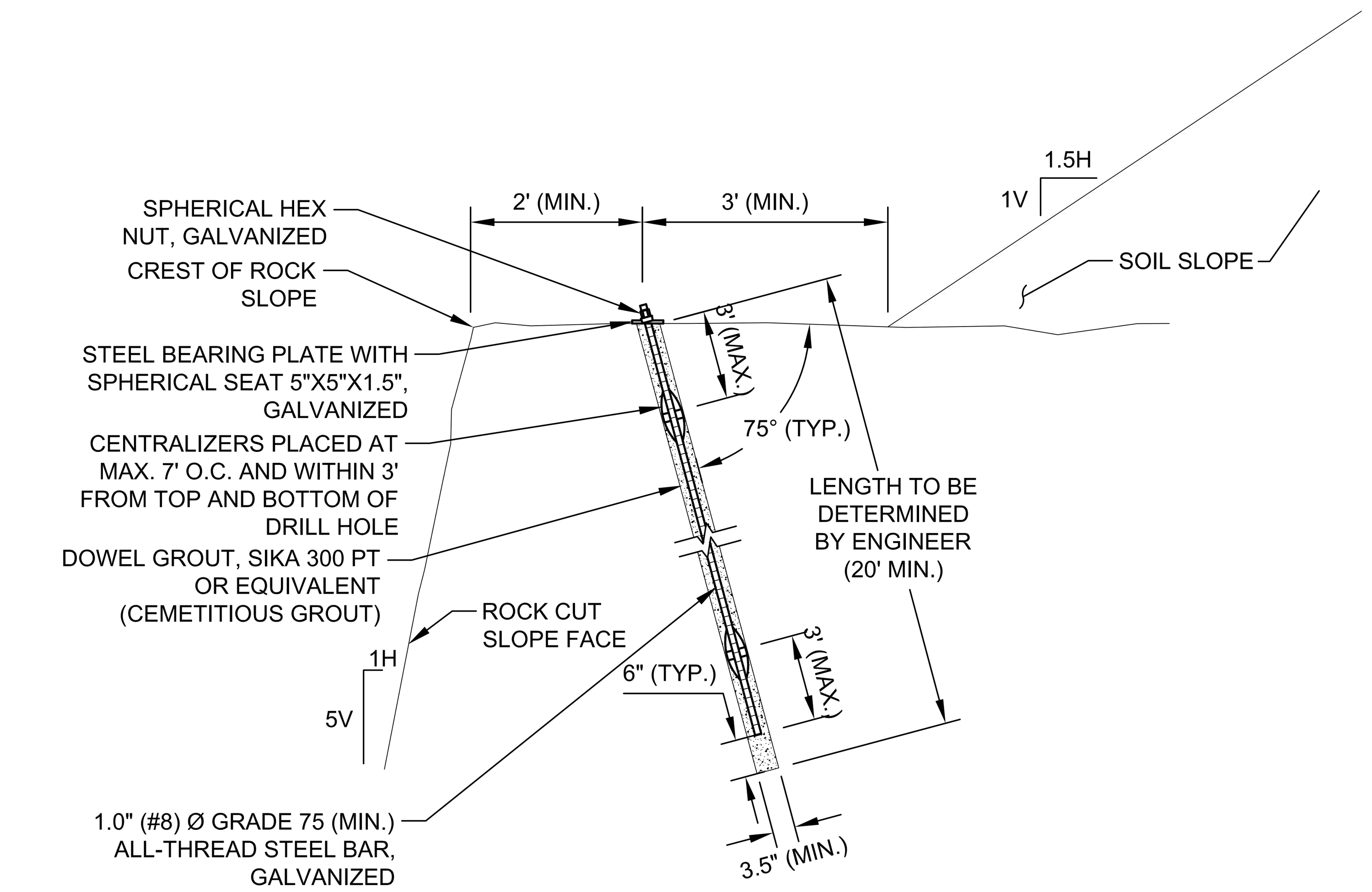
Golder Associates
Newark, New Jersey



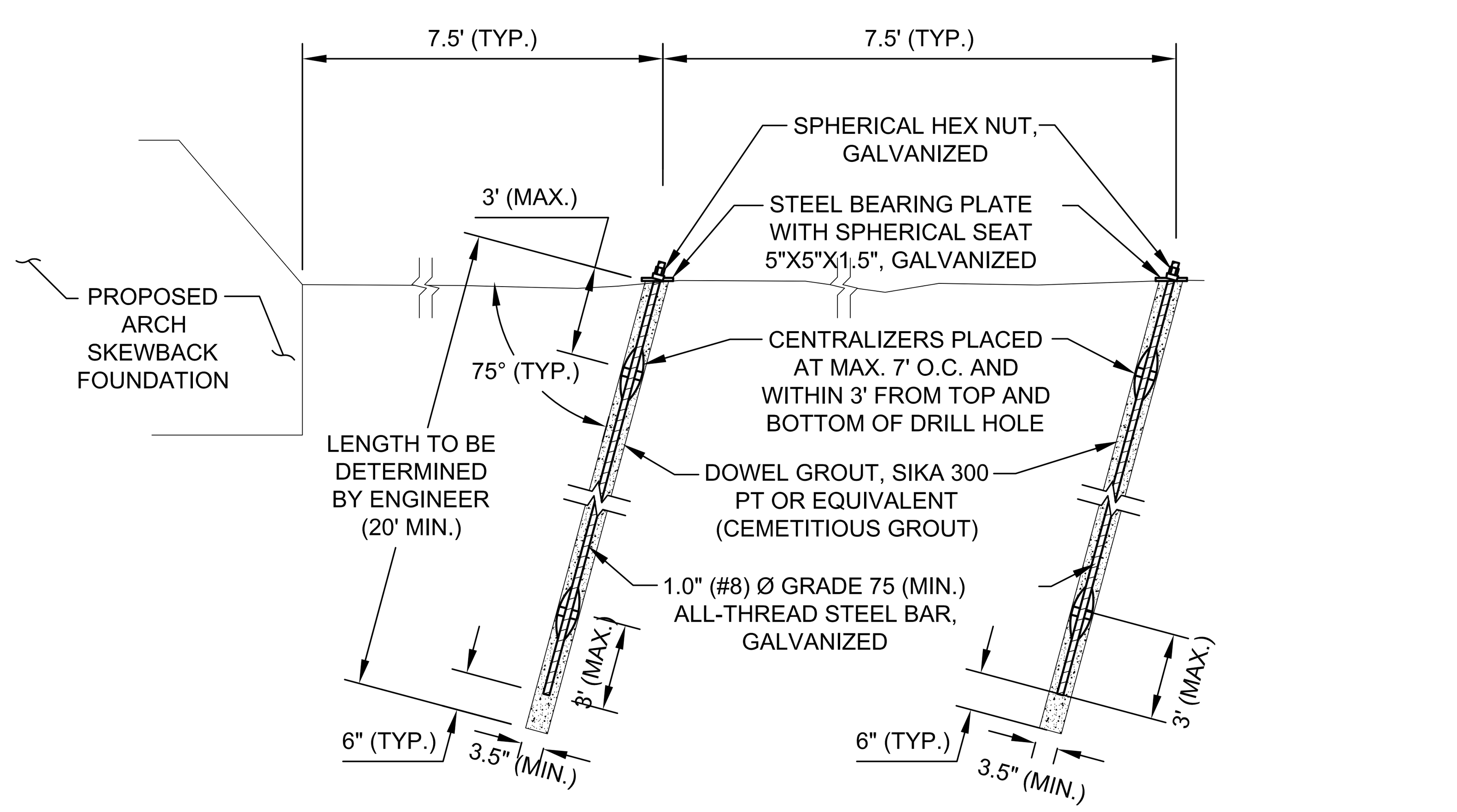
N.T.S. 1/9 TYPICAL WIRE ROPE ANCHOR DETAIL



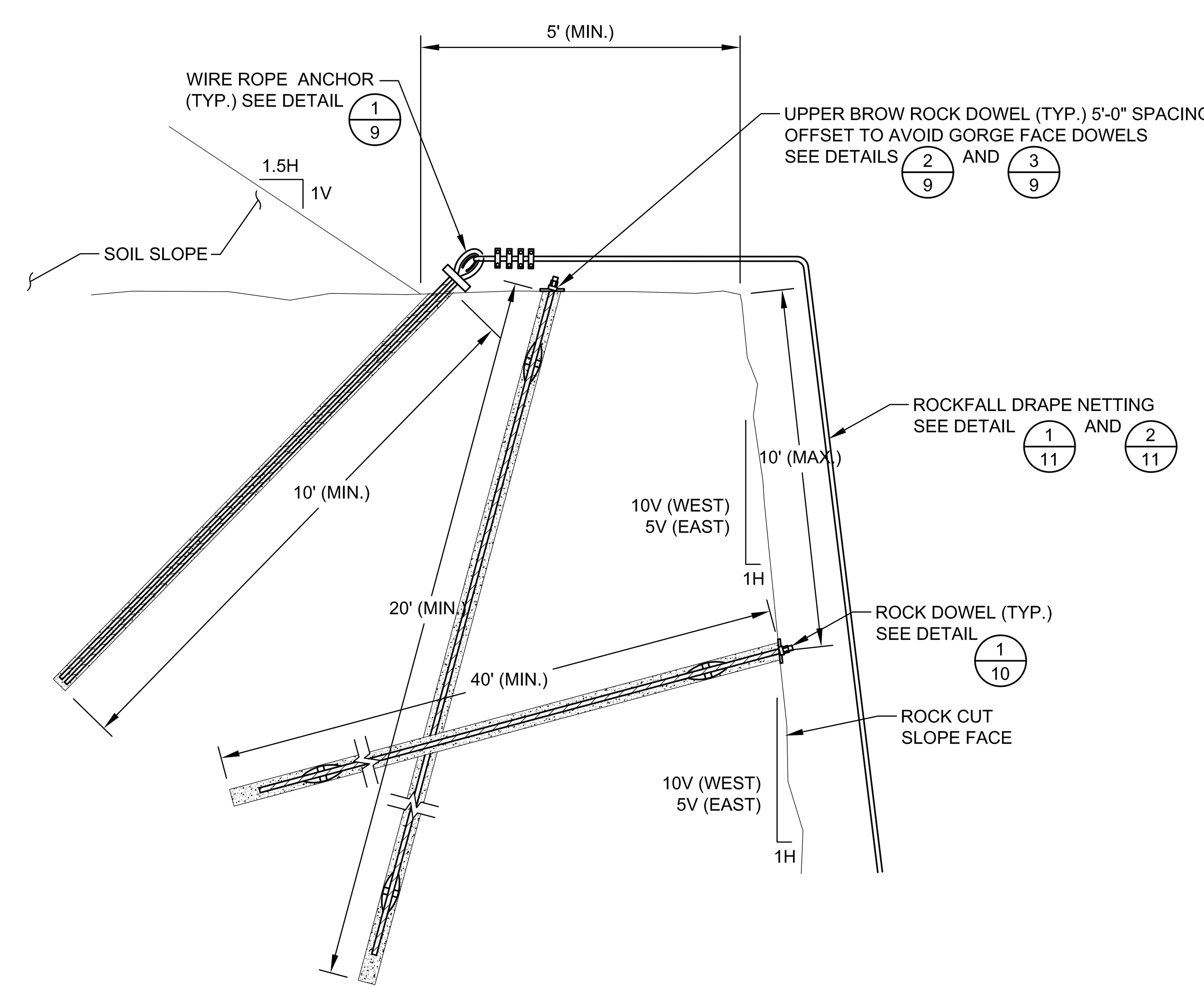
N.T.S. 2/9 TYPICAL UPPER BROW ROCK DOWEL DETAIL - WEST SLOPE



N.T.S. 3/9 TYPICAL UPPER BROW ROCK DOWEL DETAIL - EAST SLOPE



N.T.S. 4/9 TYPICAL LOWER BROW ROCK DOWEL DETAIL - WEST SLOPE



N.T.S. 5/9 TYPICAL UPPER BROW STABILIZATION DETAIL

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

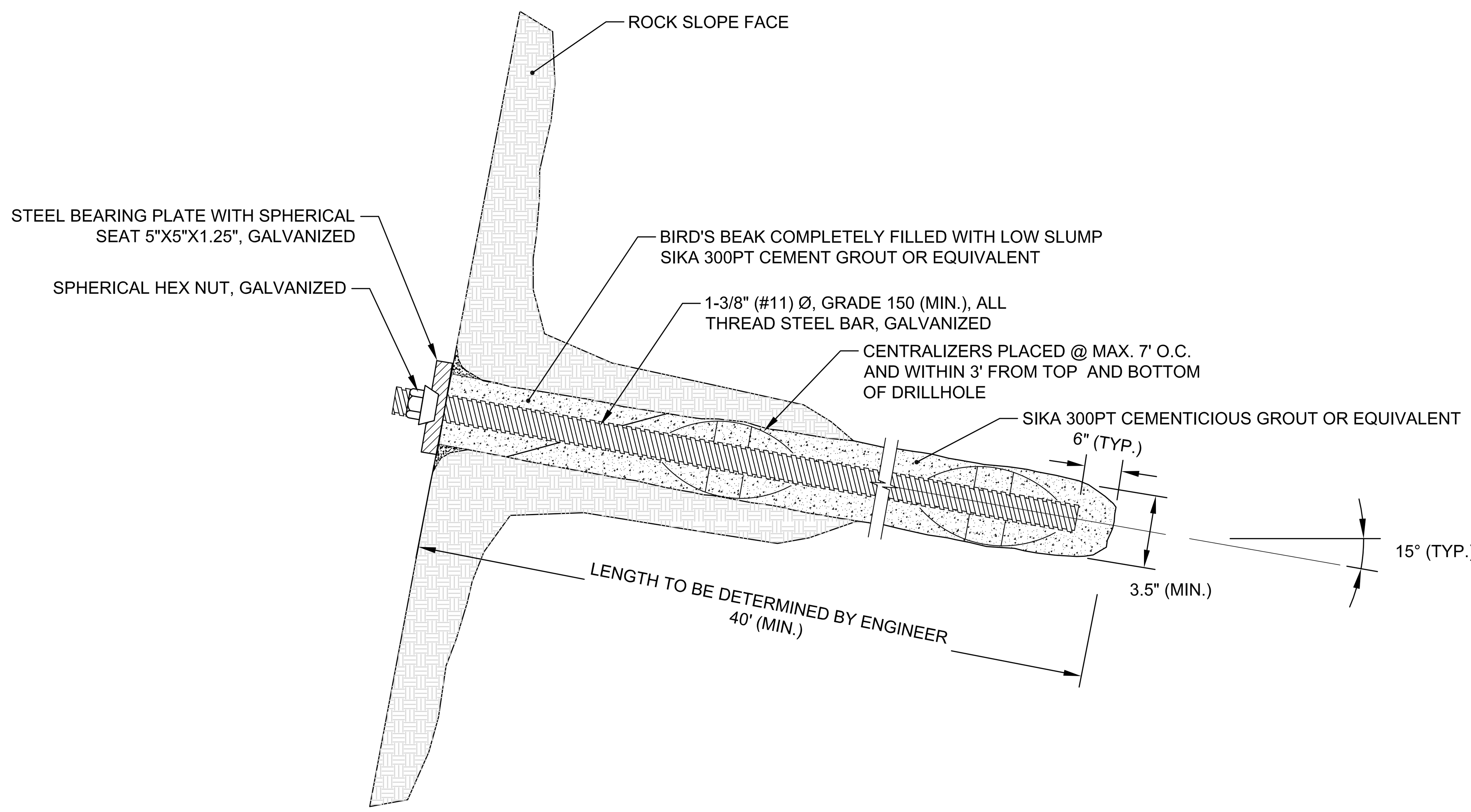
PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE
DETAIL SHEET (1 OF 4)

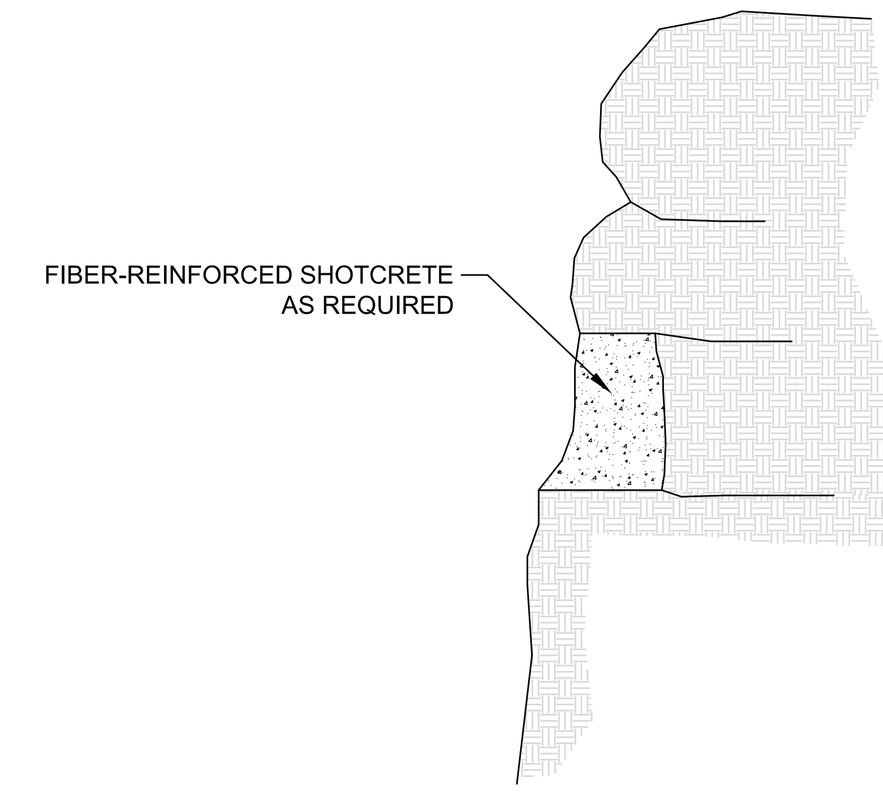
NJ Authorization #24GA28029100

Golder Associates
Newark, New Jersey

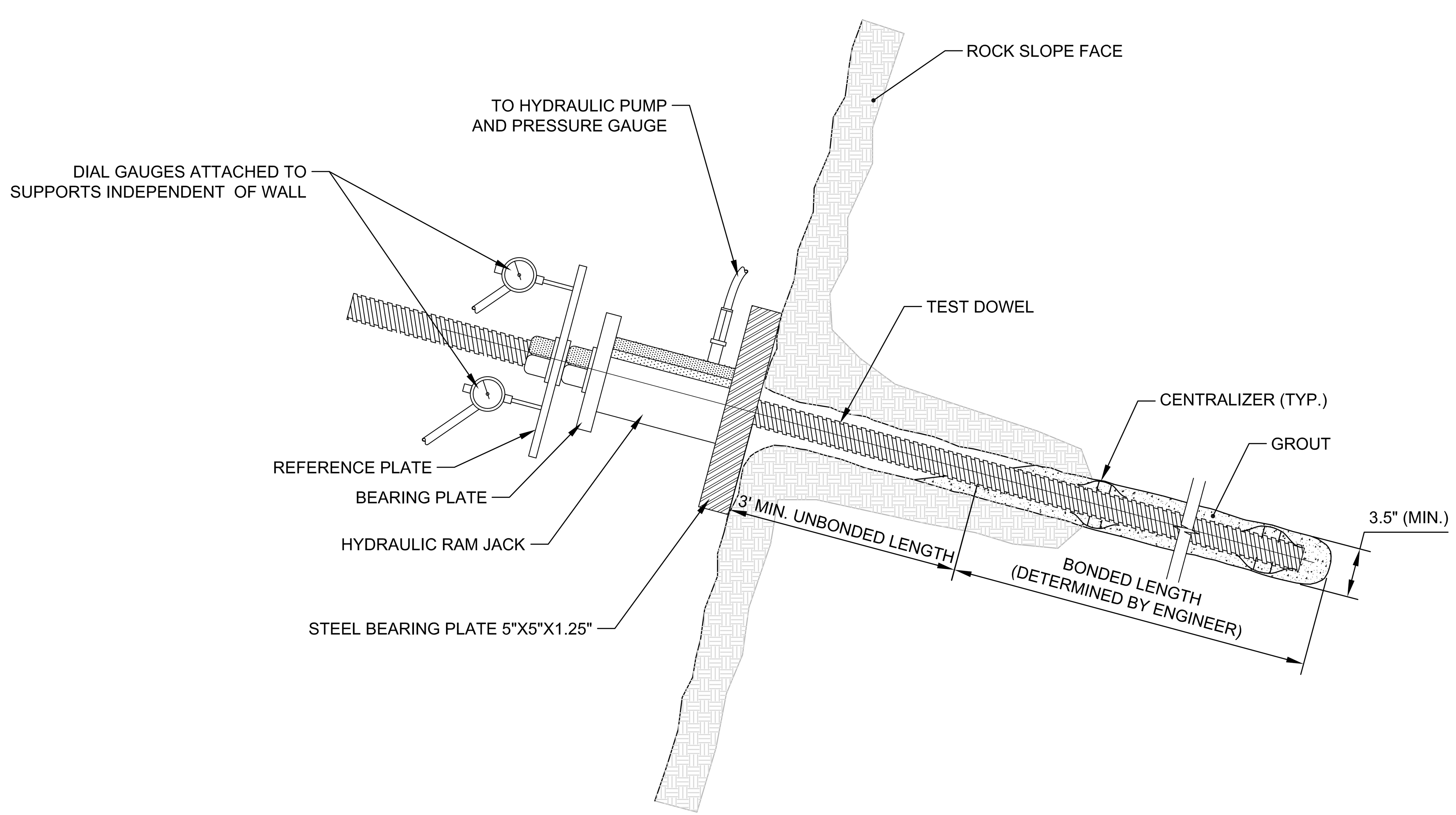
PROJECT No.	103-86509.01		FILE No.	10386509D011
DESIGN	JRS	02/08/13	SCALE	N.T.S. REV. 0
CADD	MPB	02/08/13	FIGURE 9	
CHECK	DKL	02/08/13		
REVIEW	MFM	02/08/13		



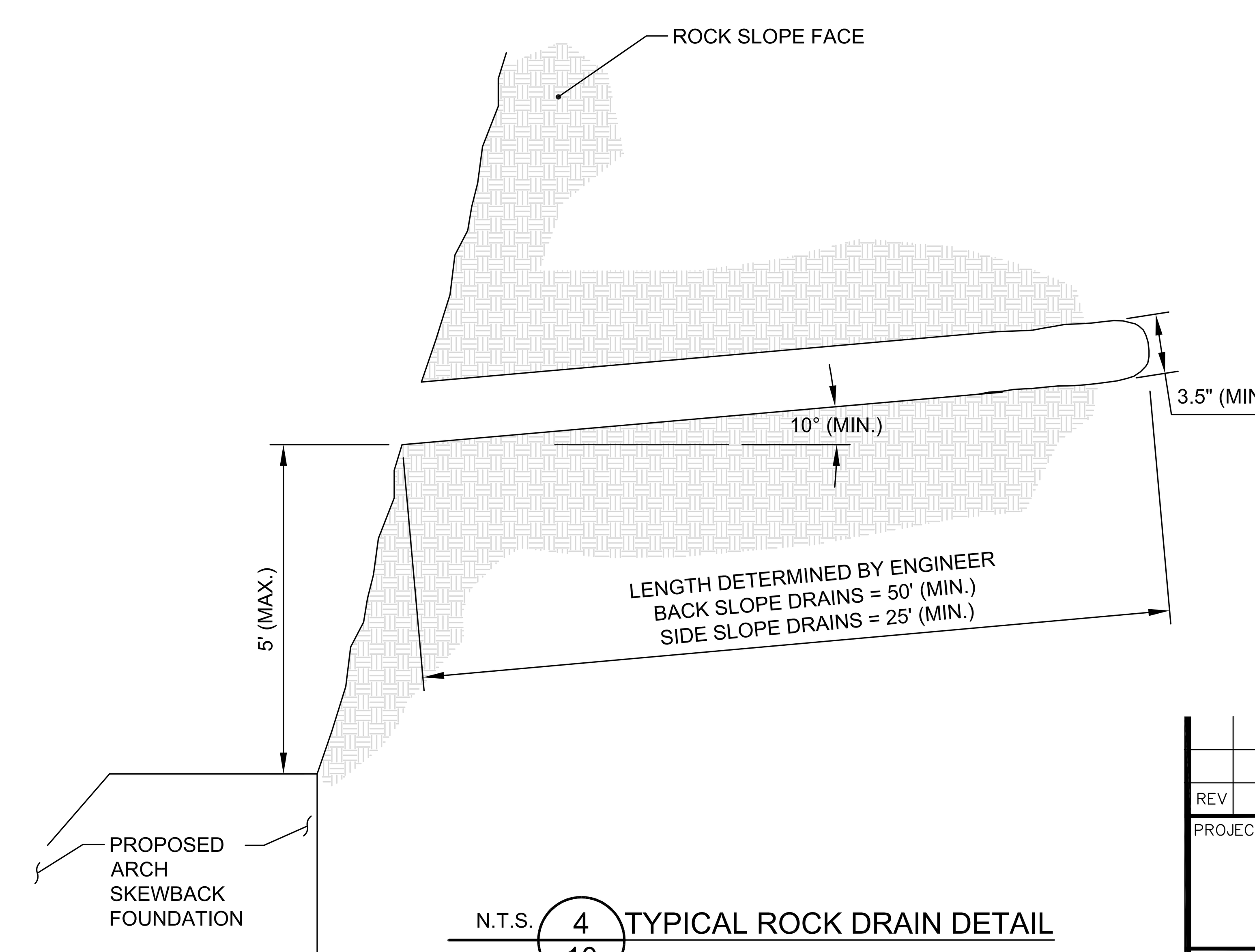
N.T.S. 1/10 TYPICAL ROCK DOWEL DETAIL



N.T.S. 2/10 TYPICAL SHOTCRETE DETAIL



N.T.S. 3/10 TYPICAL ROCK DOWEL PULL-OUT TEST DETAIL



N.T.S. 4/10 TYPICAL ROCK DRAIN DETAIL

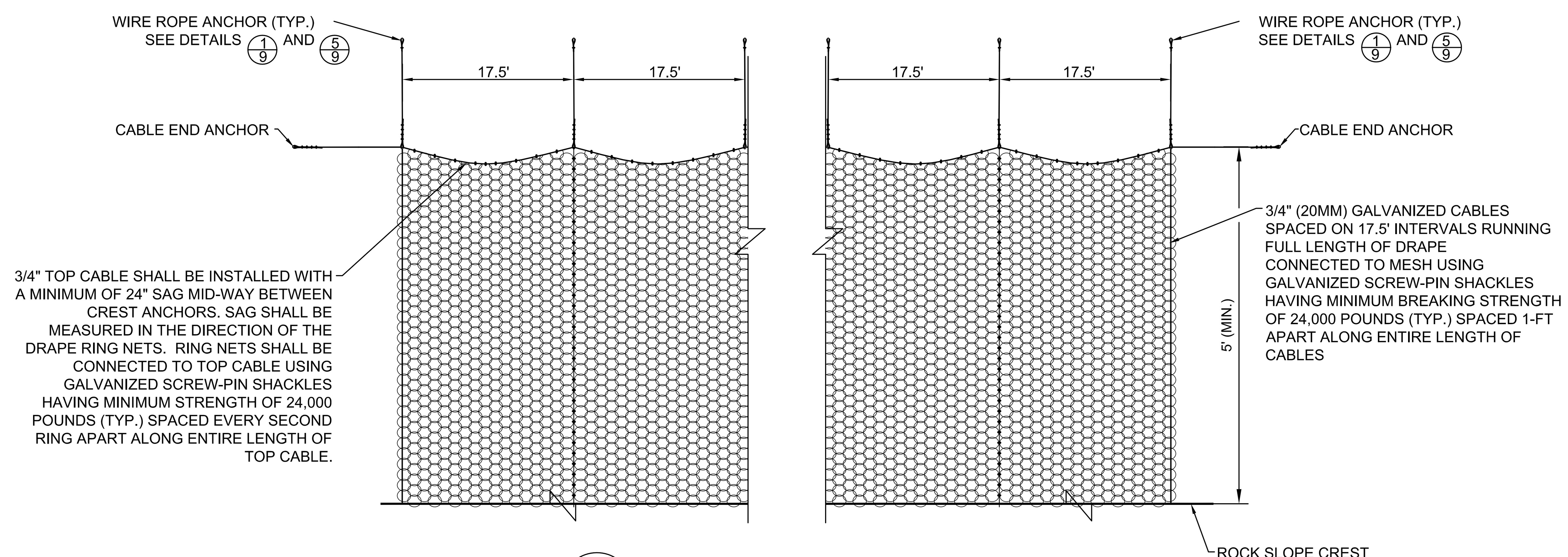
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPACEMENT PROJECT
PORTAGEVILLE, NEW YORK

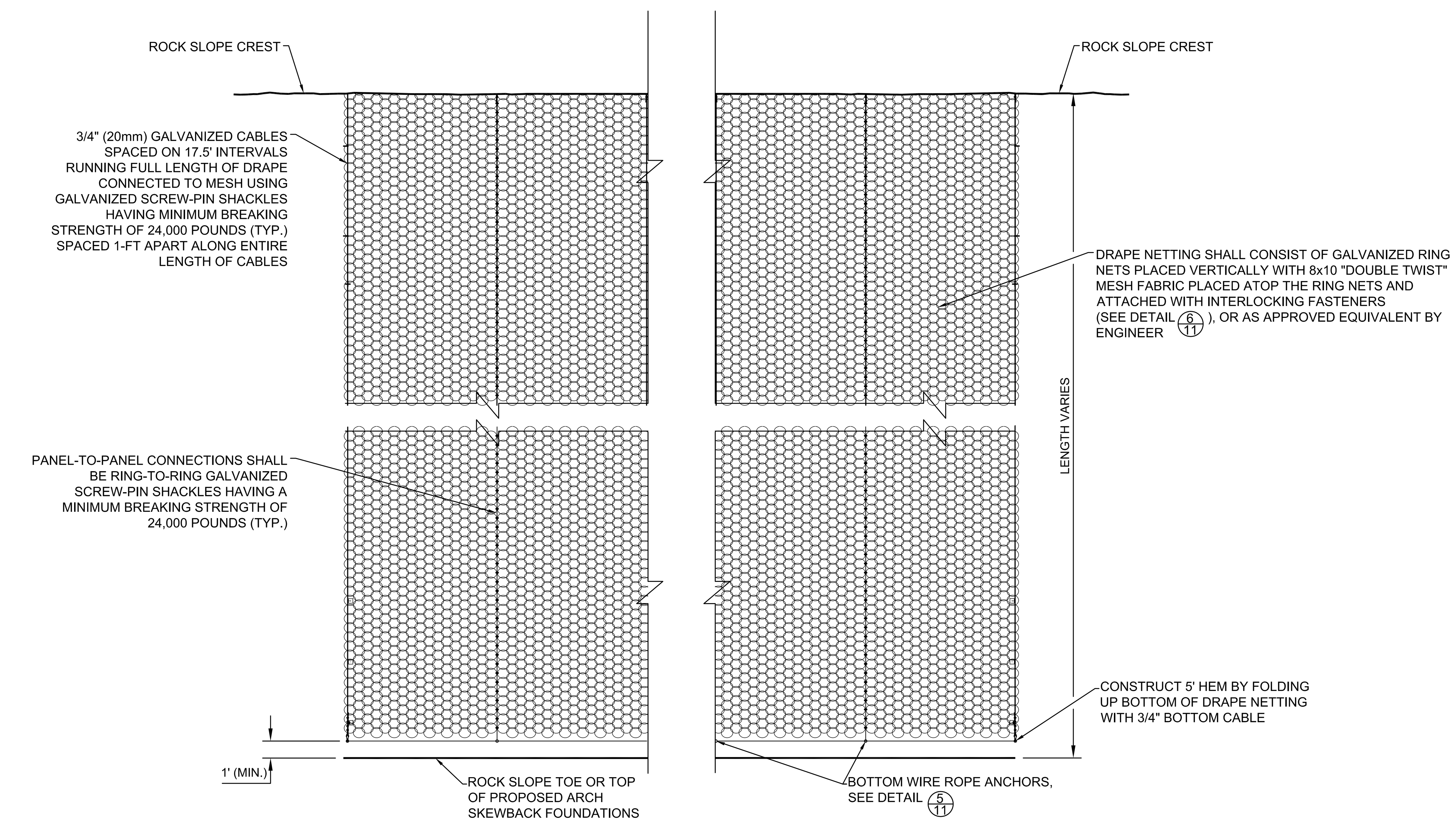
TITLE
DETAIL SHEET (2 OF 4)

NJ Authorization #24GA28029100
Golder Associates
Newark, New Jersey

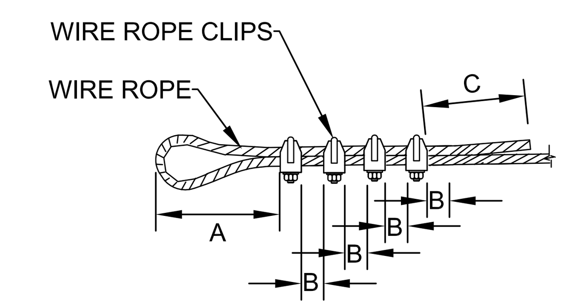
PROJECT No.	103-86509.01		FILE No.	10386509D012
DESIGN	JRS	02/08/13	SCALE	N.T.S. REV. 0
CADD	MPB	02/08/13	FIGURE 10	
CHECK	DKL	02/08/13		
REVIEW	MFM	02/08/13		



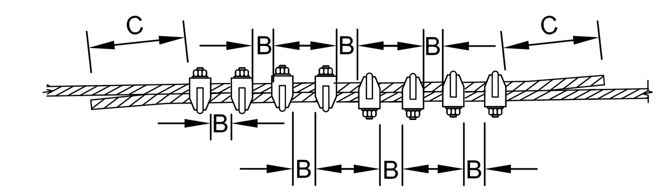
N.T.S 1/11 TOP VIEW (PLAN) OF ROCK SLOPE



N.T.S 2/11 FRONT FACE OF ROCK SLOPE

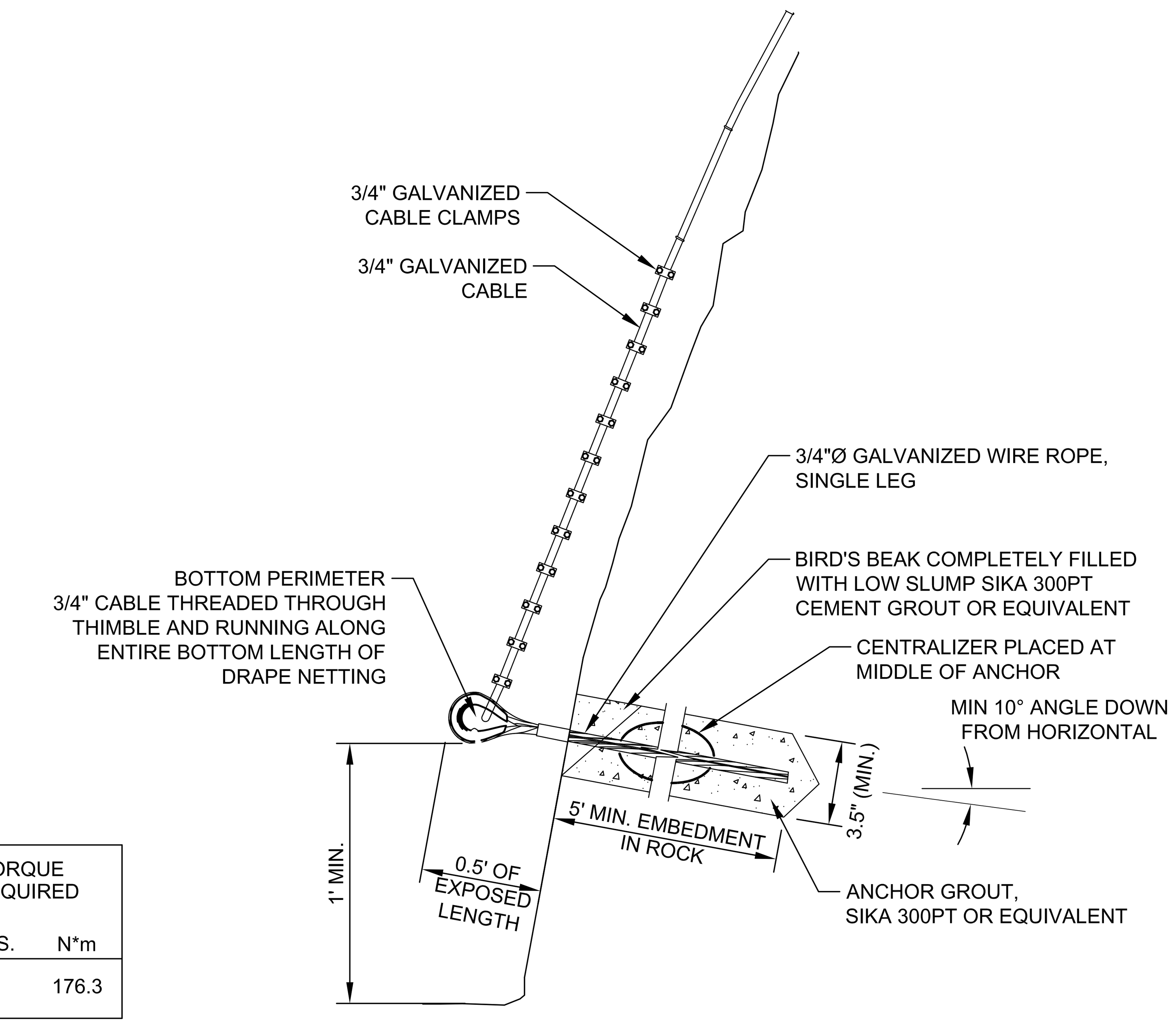


N.T.S 3/11 TYPICAL WIRE ROPE ANCHOR LOOP DETAIL

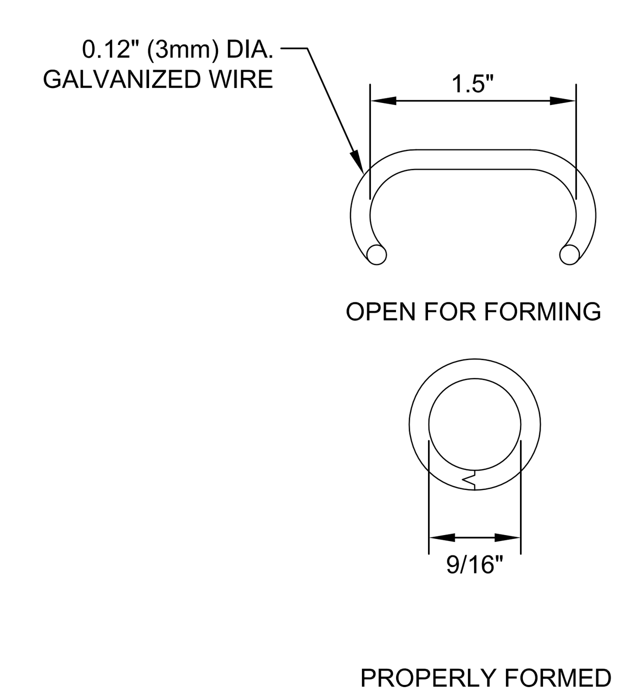


ROPE DIAMETER		MINIMUM WIRE ROPE CLIP SIZE		MINIMUM WIRE ROPE CLIP QUAN.	A		B		C		TORQUE REQUIRED	
IN.	mm	IN.	mm		IN.	mm	IN.	mm	IN.	mm	FT-LBS.	N*m
3/4	19	3/4	19	4	9	229	4	102	6	152	130	176.3

N.T.S 4/11 TYPICAL WIRE ROPE SPLICING DETAIL



N.T.S 5/11 TYPICAL BOTTOM WIRE ROPE ANCHOR DETAIL



N.T.S 6/11 TYPICAL INTERLOCKING FASTENERS

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPACEMENT PROJECT
PORTAGEVILLE, NEW YORK

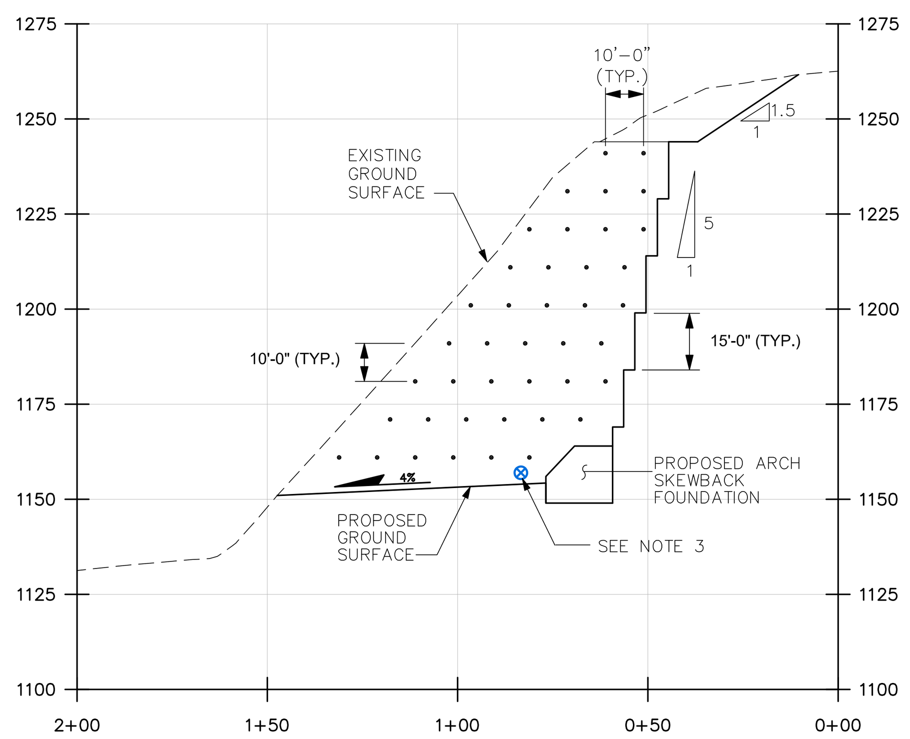
TITLE
DETAIL SHEET (3 OF 4)

NJ Authorization #24GA28029100

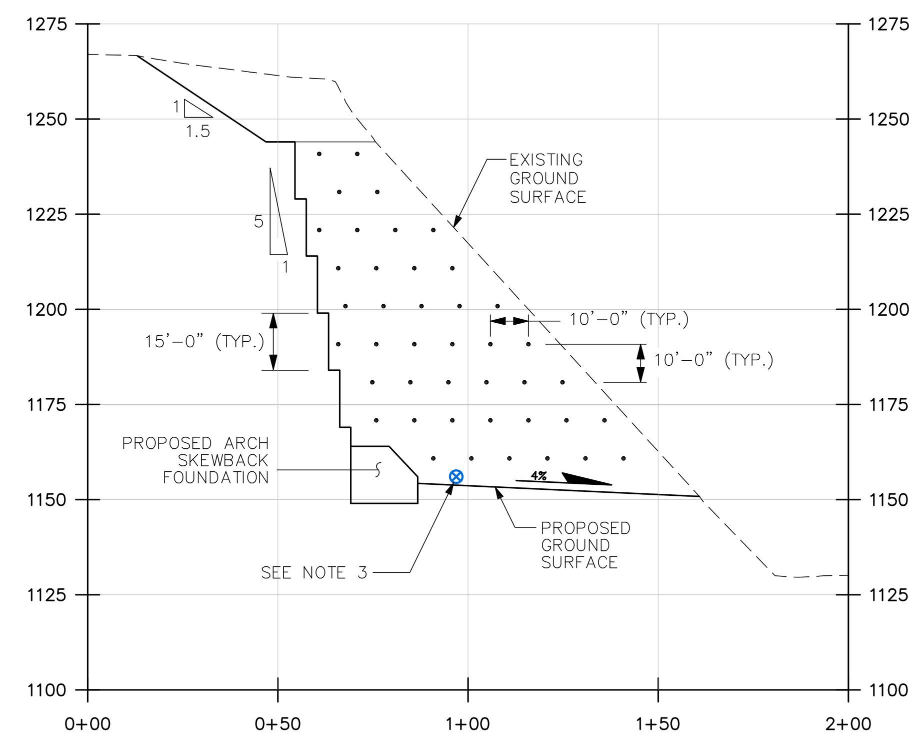
Golder Associates
Newark, New Jersey

PROJECT No.	103-86509.01		FILE No.	10386509D013
DESIGN	JRS	02/08/13	SCALE	N.T.S. REV. 0
CADD	MPB	02/08/13		
CHECK	DKL	02/08/13	**FIGURE 11**	
REVIEW	MFM	02/08/13		

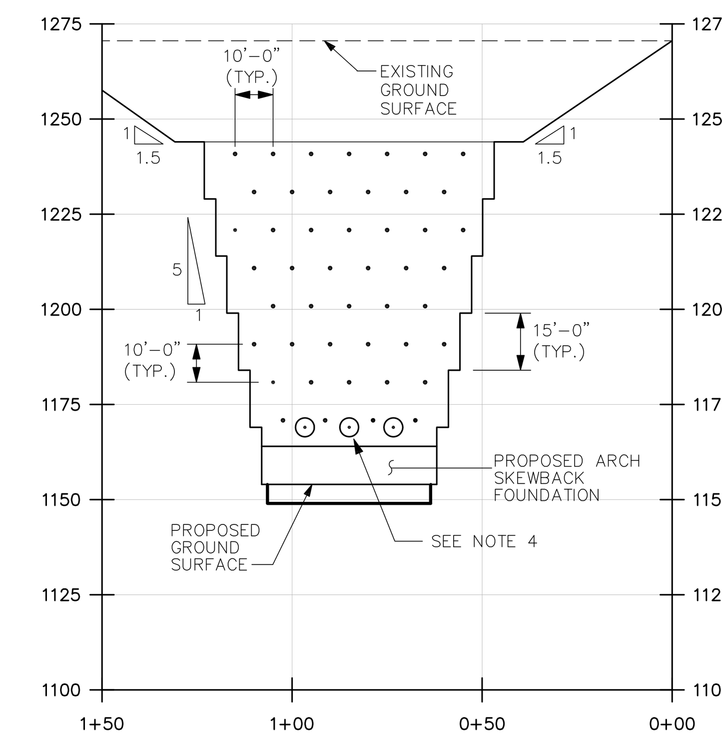
Drawing File: 10386509D013.dwg | Layout: ANSI_B_FIGURE_LANDSCAPE | Modified: 02/08/13 11:45pm | Plotted: 02/08/13 1:02pm | Plotted by: AMorales



NORTH WALL

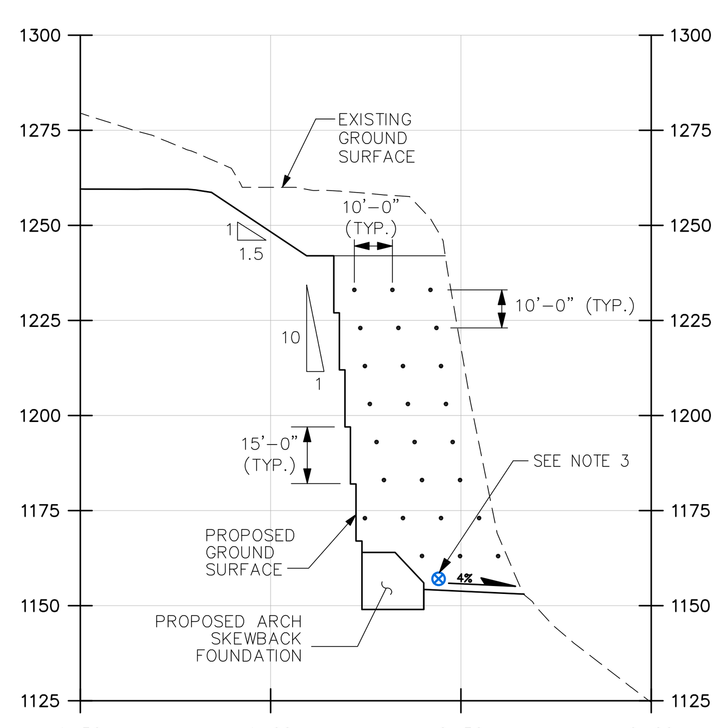


SOUTH WALL

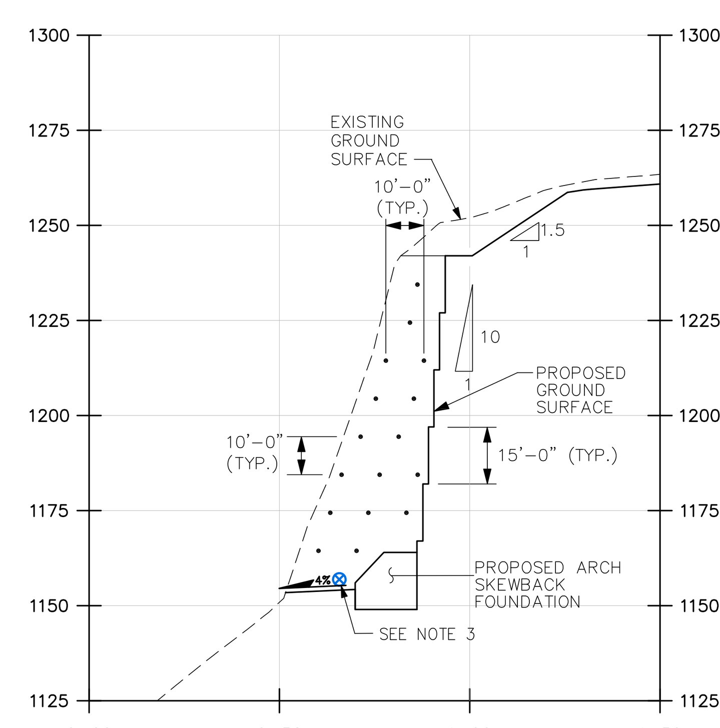


EAST WALL

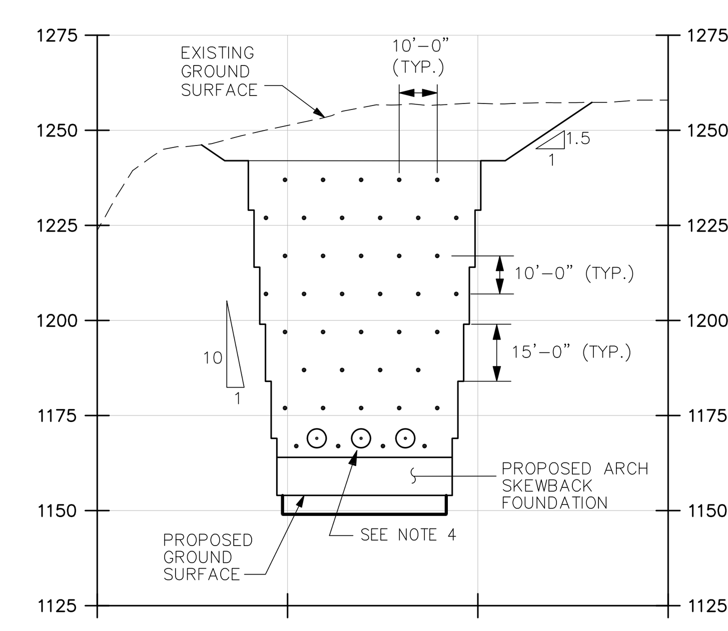
ROCK DOWEL AND DRAIN LOCATIONS - EAST SLOPE



NORTH WALL



SOUTH WALL



WEST WALL

ROCK DOWEL AND DRAIN LOCATIONS - WEST SLOPE

LEGEND

- ROCK DOWEL, SEE DETAIL 1/10
- BACK SLOPE ROCK DRAIN, SEE DETAIL 4/10
- SIDE SLOPE ROCK DRAIN, SEE DETAIL 4/10

NOTES

1.) ALL ROCK DOWEL AND ROCK DRAIN LOCATIONS AND ELEVATIONS ARE APPROXIMATE, AND WILL BE ESTABLISHED IN THE FIELD.

2.) ROCK DOWELS TO BE INSTALLED WITH 10'–0" (TYP.) STAGGERED PATTERN, AS SHOWN.

3.) 25' LONG SIDE SLOPE ROCK DRAINS TO BE LOCATED 5' (MAX.) ABOVE PROPOSED GRADE, AND PLACED TO AVOID INTERFERING WITH ROCK DOWELS.

4.) 50' LONG BACK SLOPE ROCK DRAINS TO BE LOCATED 5' (MAX.) ABOVE SKEWBACK FOUNDATION, AND PLACED TO AVOID INTERFERING WITH ROCK DOWELS.

5.) SEE FIGURES 7 AND 8 FOR LOCATIONS OF ALL SPECIFIED UPPER/LOWER BROW ROCK DOWELS AND WIRE ROPE ANCHORS.

40 0 40 80
SCALE FEET

NJ Authorization #24GA28029100

Golder Associates
Newark, New Jersey

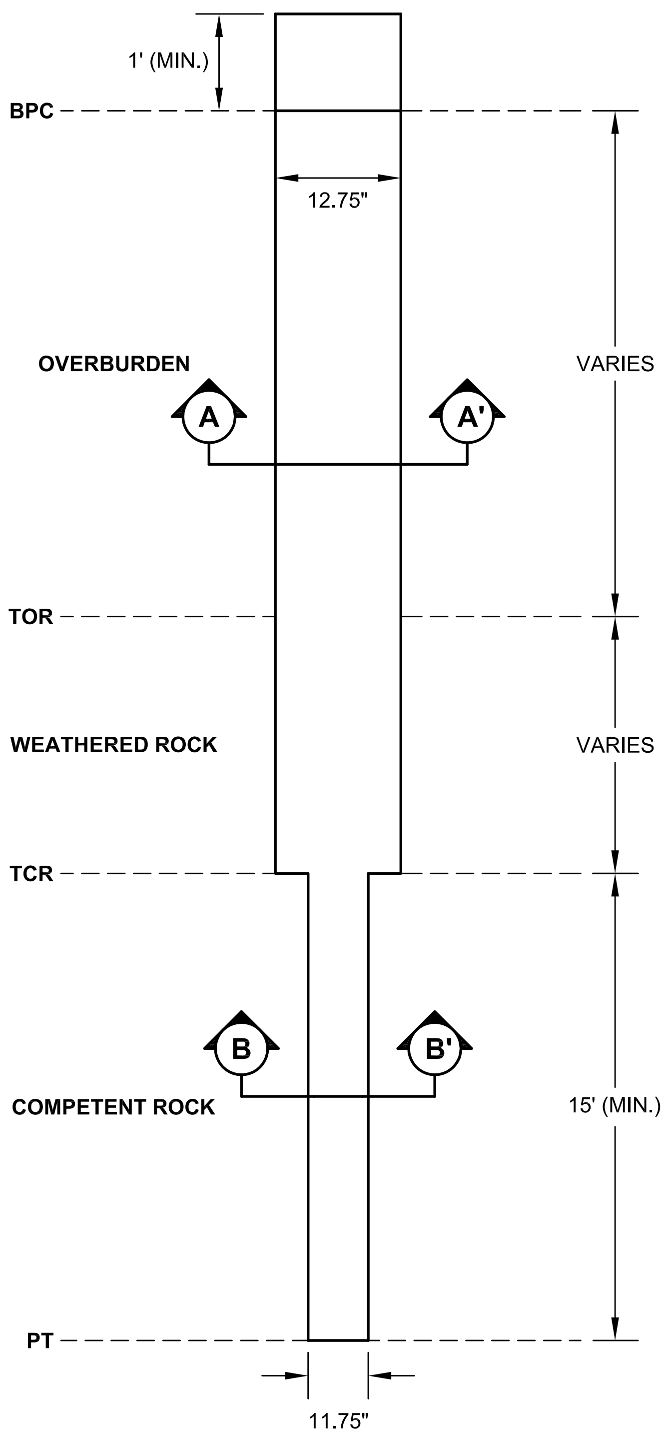
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE
DETAIL SHEET (4 OF 4)

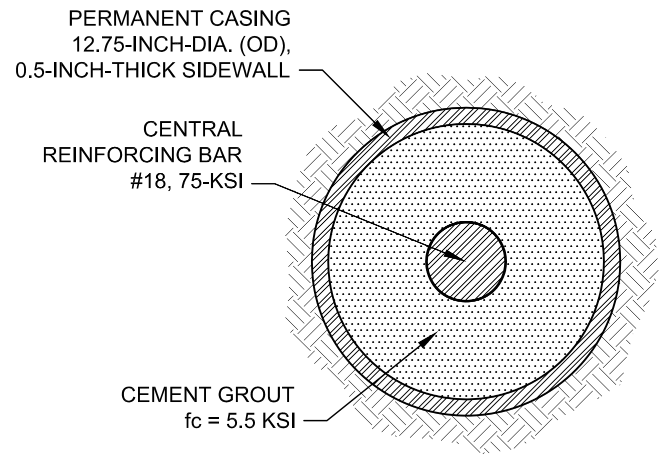
PROJECT No.		103–86509.01
FILE No.		10386509D014
REV. 0	SCALE	AS SHOWN
DESIGN	JDL	02/08/13
CADD	MPB	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

FIGURE 12



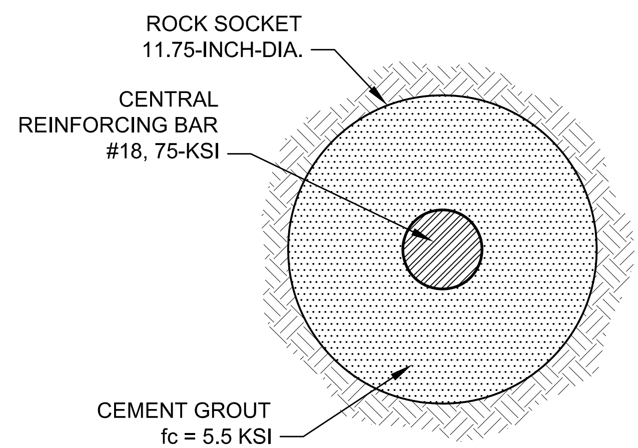
TYPICAL MICROPILE DETAIL

NOT TO SCALE



SECTION A-A'

NOT TO SCALE



SECTION B-B'

NOT TO SCALE

NOTES

- BPC = BOTTOM OF PILE CAP
- TOR = TOP-OF-ROCK
- TCR = TOP-OF-COMPETENT-ROCK
- PT = PILE TIP

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
NORFOLK SOUTHERN
HIGH BRIDGE (SR-361.66)
REPLACEMENT PROJECT
PORTAGEVILLE, NEW YORK

TITLE
MICROPILE DETAILS

NJ Authorization #24GA28029100
Golder Associates
Newark, New Jersey

PROJECT No.	103-86509.01	
DESIGN	DKL	02/08/13
CADD	RG	02/08/13
CHECK	DKL	02/08/13
REVIEW	MFM	02/08/13

FILE No. 10386509D008
SCALE AS SHOWN | REV. 0

FIGURE 13

APPENDICES